PB-239 115

AN EVALUATION OF THREE COMBINED SEWER OVERFLOW TREATMENT ALTERNATIVES

John W. Parks

Clark, Dietz and Associates

Prepared for:

National Environmental Research Center

December 1974

DISTRIBUTED BY:

NTIS
National Technical Information Service
U. S. DEPARTMENT OF COMMERCE

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

1. REPORT NO. EPA-670/2-74-079	2.	PB 239 115

4. TITLE AND SUBTITLE AN EVALUATION OF THREE COMBINED SEWER OVERFLOW TREATMENT ALTERNATIVES	5. REPORT DATE December 1974-Issuing Date 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) John W. Parks	8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Clark, Dietz and Associates-Engineers, Inc. Urbana, Illinois 61801	10. PROGRAM ELEMENT NO. 1BB034/ROAP 21ASY/TASK 132 11. ~~CONTRACT/~~GRANT NO. Grant No. 11020 FAM
12. SPONSORING AGENCY NAME AND ADDRESS National Environmental Research Center Office of Research and Development U.S. Environmental Protection Agency Cincinnati, Ohio 45268	13. TYPE OF REPORT AND PERIOD COVERED Final 3/67 - 12/73 14. SPONSORING AGENCY CODE
15. SUPPLEMENTARY NOTES	

16. ABSTRACT

An evaluation of the performance of three different combined sewer overflow treatment units is presented, with influent and effluent flow, BOD, and suspended solids data for a single-cell flow-through lagoon; a retention lagoon followed by a two-cell facultative lagoon system and effluent chlorination; and a sedimentation tank with influent chlorination. The extent of preconstruction pollution from combined sewer overflow is discussed as well as the success of partial sewer separation in reducing the strength of combined sewer overflow. The performance of the two-cell facultative lagoon as a tertiary treatment system for the existing sewage treatment plant effluent is also analyzed in detail, demonstrating a dual use for this type of combined sewer overflow treatment unit.

Each of the three treatment alternatives were found to provide substantial reduction in combined sewer overflow pollution, with the lagoon units giving the greater removal efficiencies. Specific recommendations for additional study and evaluation are offered.

Reproduced by
NATIONAL TECHNICAL
INFORMATION SERVICE
US Department of Commerce
Springfield, VA. 22151

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
*Combined sewers *Overflows *Sewage treatment Lagoons (ponds)	*Combined sewer overflows Combined sewage Retention basins Wet weather testing Preconstruction evaluation	13B

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS *(This Report)* UNCLASSIFIED	21. NO. OF PAGES 123
	20. SECURITY CLASS *(This page)* UNCLASSIFIED	22. PRICE 5.25

PRICES SUBJECT TO CHANGE

0 37022

EPA-670/2-74-079
December 1974

AN EVALUATION OF
THREE COMBINED SEWER OVERFLOW
TREATMENT ALTERNATIVES

By

John W. Parks
Clark, Dietz and Associates-Engineers, Inc.
Urbana, Illinois 61801

Project No. 11020 FAM
Program Element 1BB034

Project Officer

James H. Phillips
U.S. Environmental Protection Agency
Region V
Chicago, Illinois 60606

NATIONAL ENVIRONMENTAL RESEARCH CENTER
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

1W

REVIEW NOTICE

The National Environmental Research Center -- Cincinnati has reviewed this report and approved its publication. Approval does not signify that the contents necessarily reflect the views and policies of the U. S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

Man and his environment must be protected from the adverse effects of pesticides, radiation, noise and other forms of pollution, and the unwise management of solid waste. Efforts to protect the environment require a focus that recognizes the interplay between the components of our physical environment -- air, water, and land. The National Environmental Research Centers provide this multidisciplinary focus through programs engaged in

- studies on the effects of environmental contaminants on man and the biosphere, and
- a search for ways to prevent contamination and to recycle valuable resources.

Essentially every metropolitan area of the United States has a stormwater pollution problem, whether served by a combined sewer or a separate system. This study presents three treatment alternatives for the reduction of combined sewer overflow pollution.

A. W. Breidenbach, Ph.D.
Director
National Environmental
Research Center, Cincinnati

ABSTRACT

An evaluation of the performance of three different combined sewer overflow treatment units is presented, with influent and effluent flow, BOD, and suspended solids data for a single-cell flow-through lagoon; a retention lagoon followed by a two-cell facultative lagoon system and effluent chlorination; and a sedimentation tank with influent chlorination. The extent of preconstruction pollution from combined sewer overflow is discussed as well as the success of partial sewer separation in reducing the strength of combined sewer overflow.

The performance of the two-cell facultative lagoon as a tertiary treatment system for the existing sewage treatment plant effluent is also analyzed in detail, demonstrating a dual use for this type of combined sewer overflow treatment unit.

Each of the three treatment alternatives were found to provide substantial reduction in combined sewer overflow pollution, with the lagoon units giving the greater removal efficiencies.

This report was submitted in fulfillment of Grant Number 11020 FAM under the sponsorship of the Environmental Protection Agency.

CONTENTS

Section		Page
I	Conclusions	1
II	Recommendations	3
III	Introduction	5
IV	Pre-Construction Evaluation	13
V	Requirements for Combined Sewer Overflow Treatment	19
VI	Pollution Abatement System	25
VII	Evaluation of Treatment Unit Operation	37
VIII	Wet Weather Performance	51
IX	Discussion	81
	Appendices	89

FIGURES

No.		Page
1	Project Location & Major Drainage Pattern	10
2	Natural Data. Preconstruction Evaluation Storm No. 1	15
3	Cumulative Data. Preconstruction Evaluation, Storm No. 1	16
4	Natural Data. Preconstruction Evaluation Storm No. 2	17
5	Cumulative Data. Preconstruction Evaluation Storm No. 2	18
6	Southeast Site Treatment System	28
7	East Site Treatment System	30
8	Southwest Site Treatment System	33
9	Southeast Lagoon Influent Natural Data Storm No. 2	56
10	Southeast Lagoon Influent Natural Data Storm No. 6	57
11	Southeast Lagoon Influent Cumulative Data Storm No. 2	58
12	Southeast Lagoon Influent Cumulative Data Storm No. 6	58
13	Southeast Lagoon Effluent Natural Data	60
14	Southeast Lagoon Effluent Cumulative Data	61
15	Treatment Tank Influent Natural Data Storm No. 5	62
16	Treatment Tank Influent Natural Data Storm No. 6	64
17	Treatment Tank Influent Cumulative Data Storm No. 5	66

FIGURES

No.		Page
18	Treatment Tank Influent Cumulative Data Storm No. 6	66
19	Treatment Tank Effluent Natural Data Storm No. 5	67
20	Treatment Tank Effluent Natural Data Storm No. 6	68
21	Storm Holding Lagoon Influent Natural Data Storm No. 12	70
22	Treatment Plant Natural Data Storm No. 12	71
23	Storm Holding Lagoon Influent Cumulative Data Storm No. 12	72
24	Storm Holding Lagoon Influent Natural Data Storm No. 13	74
25	Treatment Plant Natural Data Storm No. 13	75
26	Storm Holding Lagoon Influent Cumulative Data Storm No. 13	76
27	Storm Holding Lagoon Contents & Effluent Data	76
28	Tertiary Lagoon Data	78
29	Sampling Station Location	92
30	3-Day Flow Chart For Sampling Stations	99
31	Sampling Station Location	102
32	Organism Distribution	109

TABLES

Table		Page
1	Population of Shelbyville	7
2	Precipitation Amounts & Frequencies	8
3	Original Treatment Plant Design Data	12
4	Analysis Data Storm No. 1	16
5	Analysis Data Storm No. 2	18
6	Dry Weather Sampling Program	38
7	Wet Weather Sampling Program	39
8	Southeast Lagoon Dry Weather Operation	43
9	Southwest Site 6-day Dry Weather Operation	46
10	Southwest Site 24 hr. Dry Weather Operation	48
11	Rainfall and Sampling Efforts	52
12	Wet Weather Sampling	53
13	Average BOD Loading Tertiary Lagoon System	79
14	Dry Weather Sampling Data	95
15	Dry Weather Sampling Data	96
16	Dry Weather Sampling Data	97
17	Robinson Creek Grab Sample Data	98
18	List of Occurances of Biological Organisms	104
19	Abundance of Organisms in the Unnamed Stream	105
20	Physical and Chemical Data from Unnamed Stream	106

ACKNOWLEDGEMENTS

The assistance and cooperation received from the City of Shelbyville during the period of the project and in the evaluation and preparation of this report is acknowledged. Many persons deserve mention in this respect. The Honorable William E. LeCrone, former Mayor of Shelbyville, submitted the original grant application and was appointed the first Project Director in April 1967. This responsibility passed to the Honorable James O. Finks, when he became Mayor of Shelbyville in 1969. Mr. Forrest B. Price, Commissioner of Finance and Water, City of Shelbyville, coordinated the activities between the consultant and the City of Shelbyville and Mrs. Marjorie S. Sylvester, City Clerk, responsibly monitored the administration of the business and financial affairs of the grant throughout the project.

John W. Parks, Director of Environmental Engineering, Clark Dietz & Associates, Inc. was the principal investigator.

Also deserving of special mention are Burl Thompson, Chief Operator of the Shelbyville Waste Water Treatment Plant and Jack Robinson, Chief Operator of the Shelbyville Area Water System. Their excellent cooperation and assistance, often at late hours and at their own expense, is appreciated.

Laboratory analysis were performed under the direction of John M. Bell, PhD, presently Associate Professor of Environmental Engineering, Purdue University, West Lafayette, Indiana.

A pre-construction biological stream investigation was performed by Warren U. Brigham, PhD, presently Assistant Aquatic Biologist for the Illinois Natural History Survey.

Assistance provided by Project Officers Robert Feder, National Environmental Research Center, Cincinnati, Ralph G. Christensen, Region V, Office of Research and Development and more recently by James H. Phillips, Office of Research and Development, Region V, EPA is gratefully acknowledged.

SECTION I

CONCLUSIONS

Three different types of systems for treatment of combined sewer overflow were found to be effective in reducing the measured pollutional discharges during wet weather periods in Shelbyville, Illinois. Partial separation of combined sewers was also effective in maintaining organic loading to the sewage treatment plant during periods of combined sewer overflow.

A single-cell overflow retention lagoon designed on the basis of containing the entire expected run-off from a ten-year storm of one-hour (1,610,000 cu ft - 45,500 cu m) duration was found to reduce the applied BOD and suspended solids by 73% and 86% respectively. Based on dry weather performance, a two-cell facultative lagoon and chlorination system following the overflow retention lagoon will provide significant reductions in soluble nutrients and elimination of measurable fecal coliform organisms. Suspended solids and total (unfiltered) BOD reductions across the facultative lagoon system were minimal during the evaluation period due to prolific algal growth. Cost of construction was $309,500.

A single-cell facultative lagoon with controlled discharge, designed on the basis of containing up to 600% of the five-day dry weather flow originating in the drainage district (250,000 cu ft - 7,080 cu m) was found to discharge a very low BOD effluent (6 mg/1) during periods following combined sewage influent flows. Suspended solids in the effluent were considerably higher (40/75 mg/1) with only a moderate average decrease with time. Both suspended solids and BOD were discharged in greater concentrations during periods of high influent flows due to carry-over and short-circuiting. Fecal coliform were reduced across the lagoon, but in the absence of effluent chlorination were not eliminated sufficiently to meet Illinois effluent criteria. Construction costs for the single lagoon were $39,800.

A combined sewage holding and sedimentation tank with comminutor, sludge scraper, and influent chlorination, designed for an average flow of 600% of the ultimate dry weather flow originating in the drainage district (1.6 MGD - 70 l/sec), was found to provide an average BOD and suspended solids removal of 63% and 53% respectively. No fecal coliform organisms in the tank effluent were detected. Cost of construction was $153,600.

Estimated annual operation and maintenance requirements for the three facilities are: 27 mandays for the single-cell facultative lagoon, 76 mandays for the combined holding and sedimentation tank, and 102 mandays for the overflow retention lagoon followed by the two-cell facultative lagoon and chlorination.

The occurrence and magnitude of a first flush during periods of combined sewer overflow varied with the overflow intensity and duration, suggesting that for any given system there exists a velocity-duration relationship for first flush requirements. The data further indicates that there can be a separate first flush for suspended solids and BOD, depending on flow velocity and whether existing material is being flushed out of the combined sewers, or surface material is being flushed into and through the combined sewers. The necessary combined sewer flow required to produce a BOD flush is not as great as that required to produce a suspended solids flush.

Note that the performance data collected, though felt to be accurate and reliable, is not statistically sufficient to provide an accurate appraisal of long term unit performances. Seasonal variation in run-offs and the effects upon unit performance were also not investigated.

The design criteria used to size the three treatment units was based in part on the requirements and regulations of the Illinois Sanitary Water Board (SWB).

The regulatory functions of the SWB have since been taken over by the Illinois Pollution Control Board and new design criteria have been established.

SECTION II

RECOMMENDATIONS

From the observations and data collected in evaluating the performance of the three treatment units in Shelbyville, the following recommendations can be offered:

1) Design criteria used in selecting and sizing combined sewer overflow treatment units should be based on expected run-off intensities, expected first flush characteristics, and influent sewer hydraulic capacities.

2) Bypass facilities for all units should be designed such that no bypassing occurs until treatment unit capacity is reached or exceeded.

3) Bypass or overflow structures in lagoon units should be located such that excess flows must pass through the lagoon to be bypassed.

4) Provision for influent, effluent, and bypass flow measurement should be designed as part of the treatment unit, to facilitate treatment and design evaluations.

5) Consideration should be given to algal removal from lagoon effluents if a high quality effluent is required.

6) Detention time versus BOD and suspended solids removal in retention lagoons should be more thoroughly investigated, especially if algal growth is a problem.

7) The effect of hydraulic overloading versus bypassing should be more thoroughly investigated in sedimentation tank units to determine which approach offers the best degree of treatment when design criteria are exceeded.

8) The effect of successive storms on first flush characteristics should be more thoroughly studied to determine the necessity for reserve capacities in batch-type (retention lagoon) treatment units.

9) The problem of solids accumulation in lagoon units should be considered in designing influent and effluent structures.

10) Unit performances during extended periods of operation and the effects of seasonal variations in run-off should be evaluated to establish operational efficiency on a long term basis.

SECTION III

INTRODUCTION

Within the last decade there has been an ever-increasing degree of environmental awareness. As a result many problems which were ignored, or not even recognized, have recently come to the forefront. One such problem is that of combined sewer overflow into surface waters.

Combined sewers, which handle both sanitary and storm sewage, carry such increased flows during wet weather that normal sewage treatment plant capacity is usually exceeded. When this occurs, it has often been the practice to divert or bypass excess flow. Unfortunately, all too often, this excess wastewater ends up in nearby waterways reducing or depleating the dissolved oxygen, resulting in fish kills, septic odors, and unsightly conditions.

According to a study conducted by the American Public Works Association for the Federal Water Pollution Control Administration in 1967, the United States had 1,329 municipalities with a total population of 54 million people being served either totally or partially by combined sewers. With this many sewers overflowing each time it rains, it is obvious that combined sewer overflows represent one of the major pollutional problems facing our society today.

To combat this problem strict controls and regulations concerning combined sewer overflows have been enacted by the Illinois Pollution Control Board. These actions necessitate the employment of treatment facilities or alternative methods, to reduce the potential and observed pollutional effects.

Since the idea of treating combined sewage is relatively new, it becomes important to examine in detail the characteristics of the problem and to test specific treatment alternatives. In so doing, the least expensive and most effective methods can be found and ultimately utilized in alleviating the pollutional stresses now present.

The study presented here represents one such investigation by evaluating three distinct treatment alternatives used in Shelbyville, Illinois, examining and evaluating each on the performance, and general feasibility of the particular approach.

PROBLEM DEFINITION

The principal purpose of this study is to examine and evaluate the three treatment methods and designs used in Shelbyville to control and treat combined sewer overflows.

The specific study goals included:

1. Evaluation of each unit with regard to removal efficiencies, advantages, disadvantages, and design criteria.

2. Comparisons among the separate treatment units to determine which system, if any, was superior.

3. Evaluation of the worth of partial sewer separation by investigating its affect on sewage quality, and first flush characteristics.

4. Establishment of guidelines concerning design and implementation of the three systems tested so that other communities with similar problems, could be aided in the selection and design of treatment facilities.

STUDY AREA

The City of Shelbyville, Illinois, is located at latitude 39 degrees, 24 minutes, 20 seconds; longitude 88 degrees, 47 minutes, 0 seconds. It is a rural-type community with a central business district, and is the County Seat for Shelby County. All of the streets in the town are paved, although only a portion of the streets in the town have curbs and gutters.

Shelbyville is situated in the southern edge of the "Corn Belt" across the northern and central portions of Illinois. The present land use of the area surrounding Shelbyville is predominantly agricultural in nature. The complexion of the activities in and around Shelbyville is expected to change significantly in the near future as the completion of the Shelbyville Reservoir will encourage more recreation-oriented activities.

POPULATION

The population of Shelbyville has shown a moderate average increase since 1900. The recent population history and projected growth for the City is presented in Table 1.

TABLE 1. POPULATION OF SHELBYVILLE

YEAR	POPULATION	YEAR	POPULATION
1900	3546	1950	4462
1910	3590	1960	4821
1920	3568	1970	4597
1930	3491	1980	6850 (Est)
1940	4092	1990	8500 (Est)

The completion of the Shelbyville Reservoir is expected to accelerate the growth of the City at a much more rapid rate than in the past. The Corps of Engineers has estimated that the Shelbyville area can expect approximately 4.9 million visitor days of activity per year. Projections show a maximum visitor day of about 4000 persons and an average visitor day of about 800 persons to the area.

These visitors will come to Shelbyville for recreational and sporting activities, both at the lake and along the Kaskaskia River. The reaches of the river below Shelbyville are also accessible for recreation from both the shore and from Carlyle Reservoir. The new recreation areas being developed will not only draw the large recreational population, but also attract new industries with a resulting larger permanent population.

CLIMATE

The climate in the Shelbyville area is relatively moderate. Occasionally the temperatures reach 100 degrees or higher. The winters are usually short and moderate, although temperatures below zero degrees are occasionally experienced. The average annual temperature in the area is approximately 55 degrees and the average monthly temperature ranges from a maximum of 78 degrees during July to a minimum of 30 degrees in January. An extreme low of -34 degrees was recorded in the area in 1927. The maximum observed temperature of 115 degrees was observed in the area in 1901 and again in 1936.

Maximum wind movement occurs in March and the minimum in August with the average wind velocity about 10.3 miles per hour. The prevailing winds are from the south. The mean relative humidity varies from about 59 to 86 per cent.

The average annual precipitation in the area is 40.5 inches, of which 22 per cent falls in May and June. Although rainstorm are frequent in the spring, local cellular storms also occur in July and August. Snowfall is limited to the period from October thru April and seldom covers the ground with more than a few inches at a time. The average annual snowfall amounts to about 20 inches. The mean annual runoff in Shelbyville area from records of 20 to 40 years is approximately 10 inches. Table 2 shows the recent rainfall history recorded in the Shelbyville area.

TABLE 2, PRECIPITATION AMOUNTS
(Windsor, Illinois Station, 11 Miles ENE of Shelbyville)

JAN.	FEB.	MAR.	APR.	MAY	JUN.	JUL.	AUT.	SEP.	OCT.	NOV.	DEC.	ANNUAL
2.56	2.44	3.37	3.84	4.44	5.04	3.49	3.46	3.22	3.17	3.07	2.44	40.48
1971 DATA - SHELBYVILLE DAM												
0.92	3.63	1.65	0.50	4.50	3.51E	9.35	0.18	3.07	0.69	1.02	7.64	36.66E
1972 DATA - SHELBYVILLE DAM												
NR	NR	1.83	5.57	1.83	1.94	-	5.44	4.65	-	5.38	-	-

Average Rainfall Frequency for Storm Periods (Southcentral Illinois)

Average Recurrence Interval (years)	Depth (IN) for given storm period							
	5 min.	15 min.	30 min.	60 min.	2 hr.	3 hr.	6 hr.	12 hr.
2	0.35	0.70	0.95	1.25	1.5	1.7	2.0	2.3
5	0.40	0.90	1.25	1.65	1.9	2.2	2.6	3.1
10	0.50	1.05	1.50	1.95	2.3	2.6	3.1	3.7
25	0.65	1.35	1.95	2.55	2.9	3.3	4.0	4.8
50	0.80	1.65	2.35	3.10	3.7	4.1	4.9	5.8
100	0.95	1.90	2.70	3.60	4.4	4.9	5.9	7.0

E Amount is wholly or partially estimated
NR No Record
- Missing
(in. x 2.54 = cm)

GEOLOGY

In recent geological time, the Shelbyville area has been influenced by both the Illinois and Wisconsin glacial advances, as well as the erosive action of wind and water. The most recent (Wisconsin) glacial advance was from the northeast, and covered the present site of Shelbyville where it built a substantial terminal moraine approximately three miles south and west of the city. The glacial drift encountered in the area consists mostly of Wisconsin till of variable thickness and mixed texture ranging from fine clays to irregular shaped boulders. The outwash materials are also of variable texture characterized by coarse-grained soils such as gravel, sand, and silt. Post-Wisconsin alluvial deposits are found in the bottom lands of the Kaskaskia Valley and are generally fine sand or silt except where local or intermittant current velocities have allowed deposit of coarser material. A layer of wind deposited loessial material generally overlies the glacial till but natural erosion associated with wind and water movement has removed the greater portion of this material and developed numerous deep, sharply sloping ravines throughout the area. The Pennsylvanian shale bedrock is encountered occasionally in outcrops and at depths as shallow as five feet in some locations. The bedrock varies from a soft brown weathered material to a distinct hard gray unweathered condition.

Groundwater is encountered at various depths depending on the subsurface conditions, and probably varies slightly depending upon seasonal fluctuations in precipitation and surface runoff, and the water levels in the nearby creeks. Available test boring data and well logs indicate that ground water may be encountered at numerous depths ranging from two feet to twenty-five feet below ground surface, depending on the porosity and thickness of subsurface materials.

DRAINAGE BASIN

The City of Shelbyville lies in the upper reaches of the Kaskaskia River Basin, Figure 1. The Kaskaskia River flows through the approximate center of the basin in an extremely crooked channel and has a fall of only 390 feet in more than 300 miles of its course. Two major impoundments on the Kaskaskia River have been completed in recent years; one at Carlyle (1966) and one at Shelbyville (1970), both principally for flood control and recreation.

The Shelbyville Dam is located just east of the city and the reservoir, Shelbyville Lake, extends north and is situated such that no drainage from the city enters the impounded water. Water through the dam is controlled by gates, and the minimum low flow maintained in the river at Shelbyville is 6.5 MGD. Prior to construction of the dam, the river had occasional periods of zero flow.

The city is situated west of the river in a hilly area ranging in elevation from 560 to 670 feet above sea level. Only about 10 percent of the area drains directly to the Kaskaskia River. The remainder flows generally from northeast to southwest to an unnamed stream, eventually reaching the river by way of its tributary, Robinson Creek.

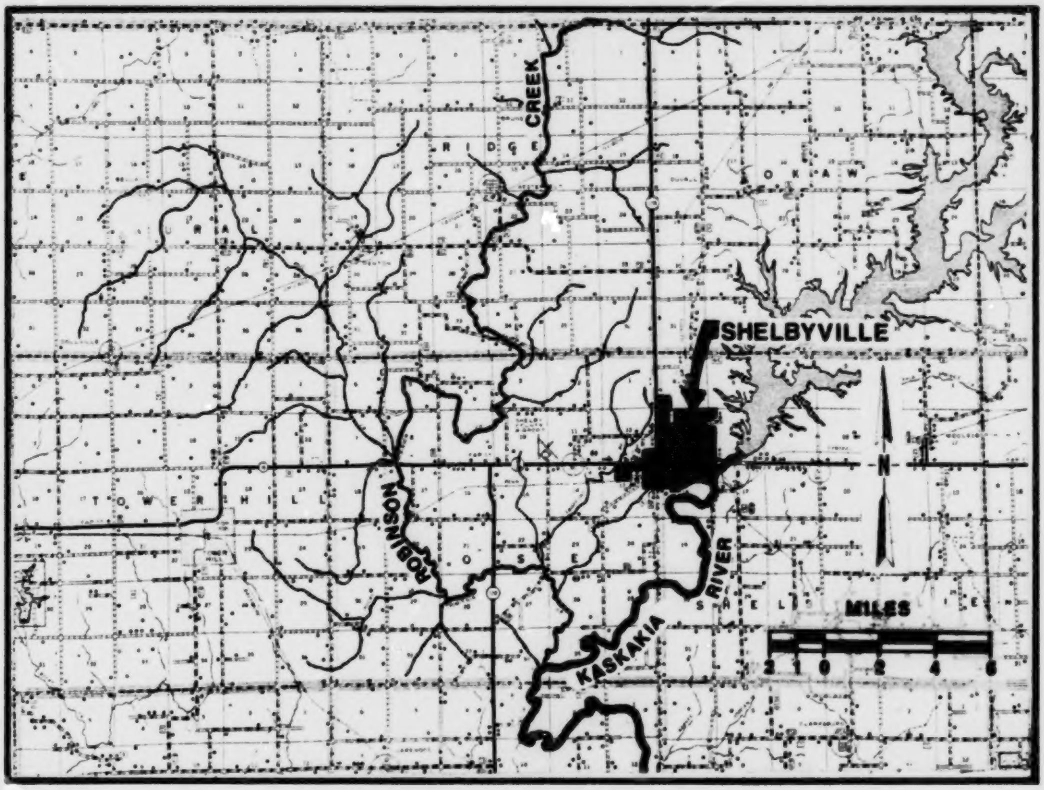

CREEK
RIDGE
OKAW
SHELBYVILLE
N
ROBINSON
TOWER HILL
ROSE
RIVER
KASKAKIA
MILES
2 1 0 2 4 6

Figure 1. Project Location and Major Drainage Pattern

DESCRIPTION OF EXISTING FACILITIES

Before 1971, Shelbyville was served by a combined sewer system that followed the natural drainage pattern of the city. This sewer system terminated with a four foot by five foot concrete interceptor sewer at the southwest corner of town. The majority of the dry weather sewage flow entered into the combined sewer system, and was diverted to a trickling filter sewage treatment plant located at the terminus of the interceptor. The remaining dry weather flow, originating on the east side of town, discharged directly into the Kaskaskia River without treatment.

During periods of storm activity the four by five foot interceptor sewer overflowed into the head waters of an unnamed stream that flows into Robinson Creek. The unnamed stream, which also receives the sewage plant effluent, meanders through farm land, and during dry weather periods, the majority of the flow is sewage treatment plant effluent. During storm periods the majority of flow in this stream was effluent from the combined sewer system in Shelbyville, as well as sewage treatment plant effluent. Prior to this abatement project, there was evidence of sludge banks in the stream, and accompanying offensive odors.

The original sewage treatment plant, completed in 1952, is located at the southwest corner of town at the termination of the four by five foot interceptor. Dry weather flow is directed by a concrete diversion structure in the interceptor to a rectangular grit chamber, through a comminutor, and into the sewage pump wet well located in the control building. Raw sewage is pumped to a 35 foot diameter primary clarifier where it then flows by gravity to a 90 foot diameter trickling filter and 35 foot final clarifier. Secondary sludge is returned to the sewage pump wet well and is the only method of recirculation employed. (The effluent recirculation pumps have been abandoned.) Primary sludge is wasted to a 35 foot floating cover anaerobic digester which is in a general state of disrepair. Sludge from the digester is wasted to sand drying beds and supernatant returned to the primary clarifier. Specific design criteria and unit sizes are shown in Table 3.

Final effluent is discharged to the unnamed stream originating at the plant site and eventually reaches the Kaskaskia River by way of its tributary, Robinson Creek, about 4 miles southwest of the plant.

TABLE 3
ORIGINAL TREATMENT PLANT DESIGN DATA

Process Type:	Single stage, standard rate trickling filter
Design Flow:	0.85 MGD
Grit Chamber:	25 ft linear channel type with proportional flow weir
Comminutor:	Rotating drum type Average Flow: 0.85 MGD Maximum Flow: 2.25 MGD
Raw Sewage Pumps:	Three units with automatic control Capacities: 350 gpm, 600 gpm, 900 gpm
Primary Clarifier:	Circular with mechanical sludge and scum removal. Diameter: 35 ft. Volume: 53,600 gal. Sidewater depth: 7 ft. Overflow weir: 110 ft.
Trickling Filter:	Circular with rotary distributor Diameter: 90 ft. Media volume: 47,300 cf Media depth: 7.5 ft. Surface area: 6,300 ft.
Final Clarifier:	Circular with mechanical sludge removal Diameter: 35 ft. Volume: 50,000 gal. Sidewater depth: 6.5 ft. Overflow weir: 110 ft.
Anaerobic Digester:	Circular with floating cover Diameter: 35 ft. Sidewater depth: 19 ft. Volume: 20,500 cf (167,600 gal.) Heat exchanger: 170,000 BTu/hr Recurculation pump: 150 gpm Primary sludge pumps (2): 75 gpm

NOTE:

gal. x 3.785 = 1
ft. x 0.305 = m
cu.ft. x 0.028 = cu.m

SECTION IV

PRE-CONSTRUCTION EVALUATION

A pre-construction evaluation program was undertaken to show (a) the magnitude of pollution resulting from by-passing untreated combined sewage, and (b) the applicability of the basic design criteria selected for construction of the combined sewage treatment facilities.

This sampling and measurement program included both dry and wet weather investigations, as well as a biological stream investigation. A detailed description of the pre-construction sampling program is given in Appendix I.

The dry weather flow investigation of the unnamed stream adjacent to the sewage treatment plant indicated that there was a generally increasing improvement of the water quality as distance downstream from the plant increased. This was especially apparent by the reductions in coliform, BOD and total volatile residue.

The biological stream investigation report, Appendix II, also showed a general improvement in stream quality at successive distances downstream from the treatment plant. The downstream increase of less tolerant organisms and the disappearance of anaerobic sludge deposits in the stream bed are significant indications of stream quality improvements.

The general physical and chemical character of the water indicated that the stream was subject to periodic "slugs" of dissolved substances; a condition apparently directly related to the combined sewage by-passing at the treatment plant. Although the data collected during this investigation indicated a definite degree of pollution, the physical and chemical nature of the stream was within the limits of tolerance of most organisms commonly found in this region.

The wet weather flow investigation was performed to provide an indication of both the nature of the combined overflows as well as relative intensity of the pollution load with time. The sampling was conducted only at the principal combined sewer outfall, as this location will discharge the greatest amount of combined sewage and was felt to be a good "worst case" analysis. Data and results for the two storm flows that were sampled are shown in Tables 4 and 5, and Figures 2 through 5.

The data shows that the untreated sewage discharged at the combined sewer outfall is a significant source of pollution of the unnamed stream adjacent to the sewage treatment plant. Figures 3 and 5 illustrate how the pollution load was discharged during a high intensity and low intensity storm.

It can be seen from an evaluation of the data that the first flush is related to the relative intensity and duration of the storm. For a low intensity storm, the first flush is of longer duration, as indicated by the slopes of the mass diagram curves. For a higher intensity storm, the slopes increase indicating that the furst flush occurs in less time.

This data was used to substantiate the choice of a 10 year design storm of one hour duration, as the maximum amount of treatment to be provided to combined sewer overflows.

Certainly storms of less intensity and longer duration might produce flows in excess of the design storm; however, this is not of major concern since most of the pollution load resulting from by-passed flow is delivered at the beginning of a storm within a time span during which the total flow will be effectively treated.

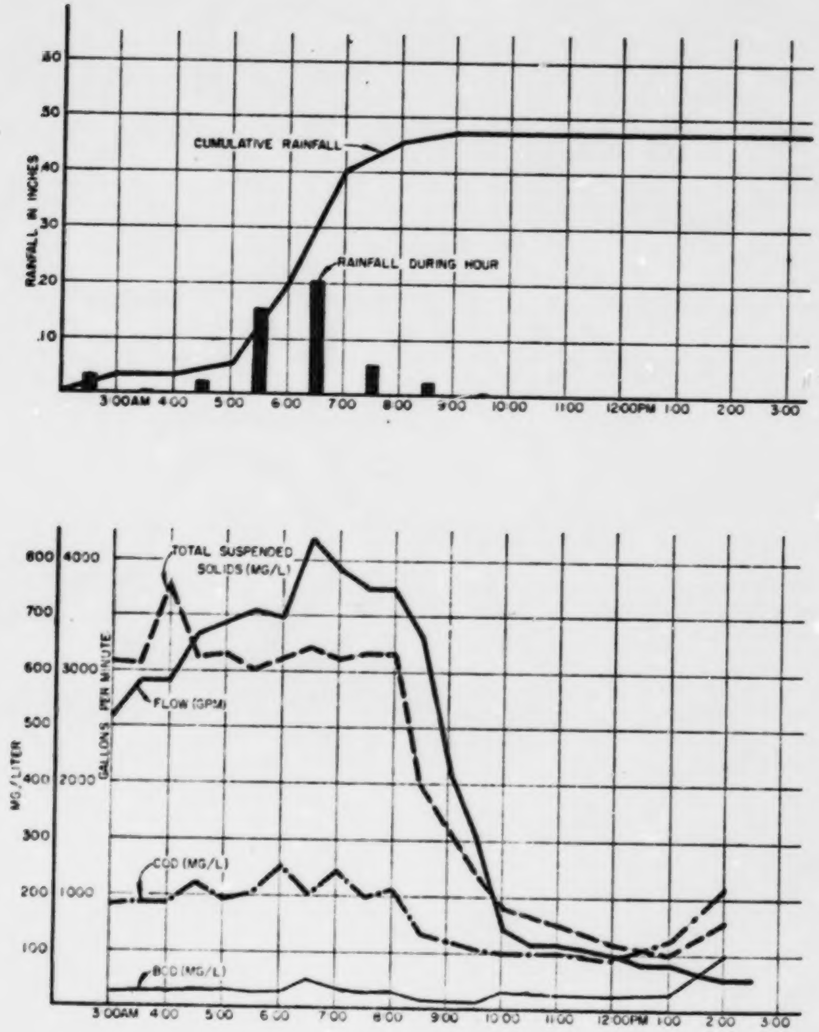


Figure 2. Natural Data, Pre-construction Evaluation Storm No. 1, January 23, 1969 (GPM x 0.063 = l/sec; inch x 2.54 = cm)

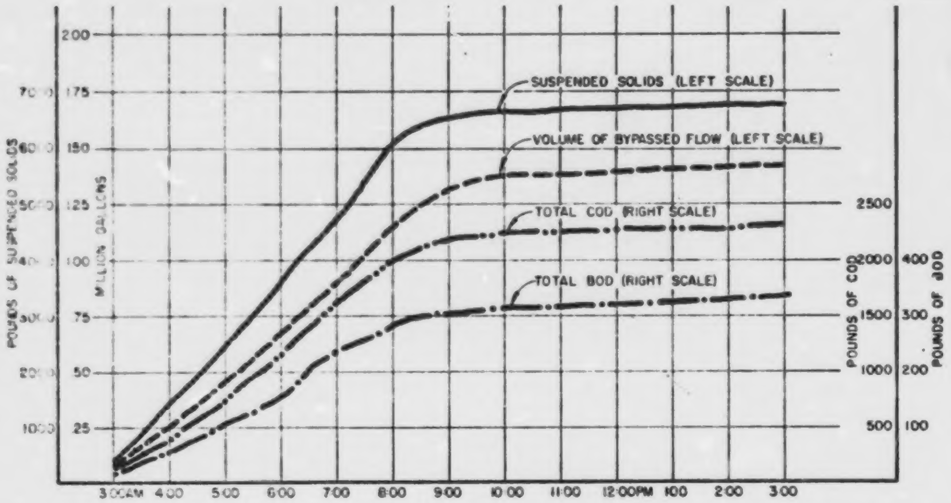


Figure 3. Cummulative Data, Pre-construction Storm No. 1

TABLE 4

ANALYSIS DATA

STORM NO. 1 Jan. 23, 1969

Sample No.	Time of Sample	pH	BOD mg/l	COD mg/l	Total Coliform No./100ml	Settleable Solids ml/l	SOLIDS (mg/l) Total Susp. Solids	Volatile Susp. Solids	Residue on Evaporation	Total Volatile Residue
1	3:00A.M.	6.8	24	180	$1.5x10^7$	1.7	617	137	806	203
2	3:30	6.8	26	186	$5.6x10^6$	1.6	615	133	841	216
3	4:00	6.9	29	187	$1.3x10^7$	2.1	756	150	858	191
4	4:30	6.9	29	220	$6.5x10^6$	2.6	628	156	851	178
5	5:00	6.8	30	195	$2.5x10^6$	1.6	633	119	829	234
6	5:30	6.9	24	210	$4.0x10^6$	2.0	605	121	870	235
7	6:00	6.8	26	255	$5.5x10^6$	1.6	628	114	835	188
8	6:30	6.8	50	205	$1.3x10^7$	1.9	647	114	829	212
9	7:00	6.8	34	245	$1.7x10^6$	2.0	626	101	820	200
10	7:30	6.8	23	200	$1.2x10^7$	1.6	635	111	874	217
11	8:00	6.6	28	215	-----	1.5	639	121	887	206
12	8:30	6.9	14	135	$4.5x10^6$	0.9	401	71	675	157
14	9:30	7.0	12	110	$3.2x10^6$	0.5	246	47	575	145
16	10:30	7.1	30	101	$4.5x10^6$	1.3	181	41	535	144
18	11:30	7.1	29	103	$2.3x10^6$	1.3	153	34	565	130
20	12:30P.M.	7.2	25	90	$1.8x10^6$	1.0	118	19	545	143
22	1:30	7.0	30	123	$4.5x10^6$	1.5	101	27	549	120
24	2:30	6.7	102	220	$4.6x10^6$	0.8	157	77	668	203

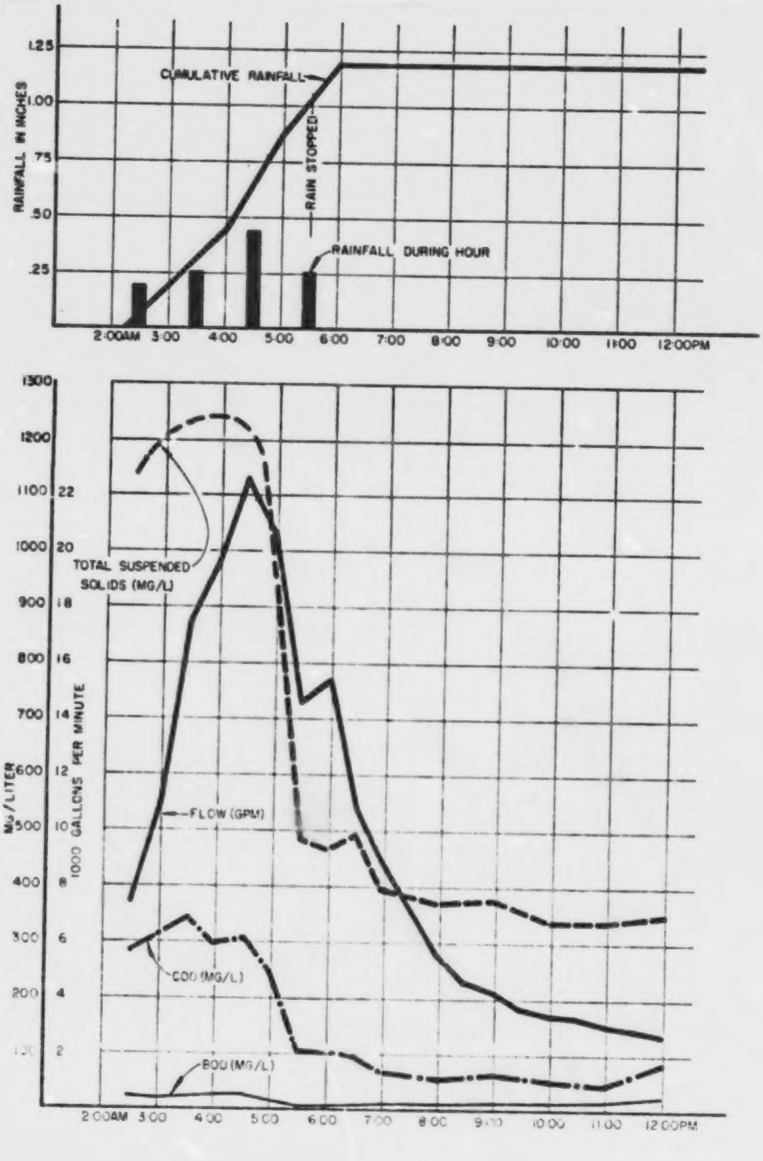


Figure 4. Natural Data, Pre-construction Storm No. 2
April 9, 1969
(gpm x 0.063 = l/sec; inch x 2.54 = cm)

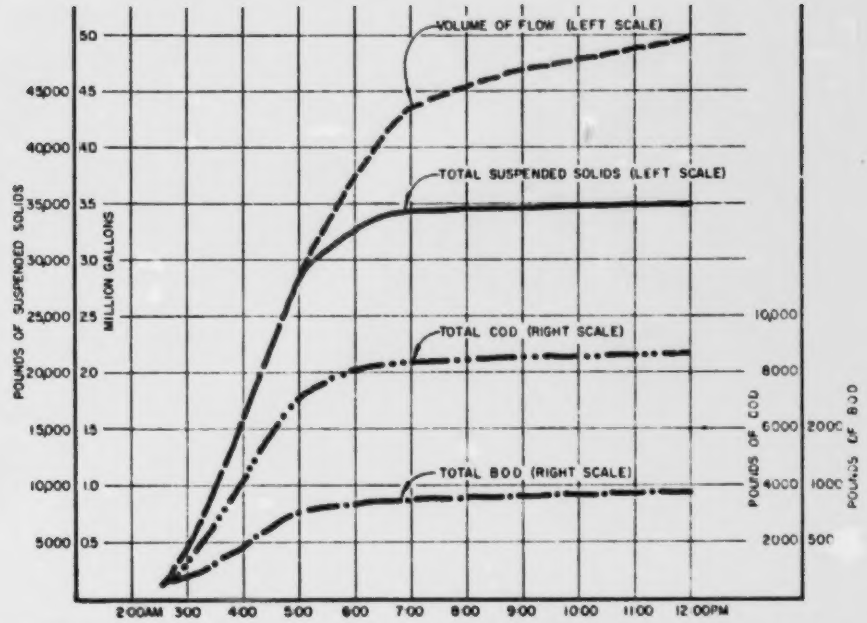


Figure 5. Cummulative Data, Pre-construction Evaluation

TABLE 5

ANALYSIS DATA
STORM NO. 2 April 9, 1969

Sample No.	Time of Sample	pH	BOD mg/l	COD mg/l	Total Coliform No/100ml	Settleable Solids ml/l	SOLIDS (mg/l) Total Susp. Solids	Volatile Susp. Solids	Residue on Evaporation	Total Volatile Residue
1	2:30 A.M.	7.0	23	286	6.5×10^6	4.0	1133	163	1238	208
2	3:00	6.9	20	314	6.3×10^6	4.2	1209	175	1369	239
3	3:30	6.8	22	345	8.4×10^6	3.6	1251	167	1287	152
4	4:00	6.9	23	295	3.0×10^6	4.4	1240	166	1245	158
5	4:30	7.1	25	309	1.8×10^6	5.4	1216	160	1293	192
6	5:00	6.9	18	243	3.8×10^6	2.4	963	117	1137	144
7	5:30	7.0	9	109	2.3×10^6	1.2	445	71	557	71
8	6:00	7.1	8	100	3.0×10^6	1.1	417	52	557	92
9	6:30	7.0	10	91	2.4×10^6	0.8	391	49	580	90
10	7:00	7.0	12	67	1.3×10^6	1.0	226	29	477	84
12	8:00	6.9	10	53	2.2×10^6	0.5	169	24	452	82
14	9:00	7.1	13	67	2.0×10^6	0.8	128	27	509	133
16	10:00	7.1	12	50	2.3×10^6	0.6	94	19	523	188
18	11:00	7.0	16	48	1.8×10^6	0.8	99	16	535	199
20	12:00P.M.	7.1	25	81	1.0×10^6	1.5	103	33	569	220

SECTION V

REQUIREMENTS FOR COMBINED SEWER OVERFLOW TREATMENT

Final design of the combined sewer overflow treatment system constructed in Shelbyville was completed in January, 1968, and at that time, the requirements and guidelines for treatment of combined sewer overflow were not as completely formulated as they are today. The treatment requirements then in effect, and used in the design of the facilities, were based on the Illinois Sanitary Water Board Rules and Regulations, SWB-14, Water Quality Standards for Intrastate Waters. The applicable portions of these rules and regulations are reproduced below in somewhat abbreviated form.

SWB-14 Rule 1.08 WATER QUALITY STANDARDS

Section 10. Treatment Requirements & Effluent Criteria

a. All municipal or industrial facilities for treatment of deoxygenating waste shall provide at least secondary biological treatment, or advanced waste treatment, adequate to reduce the organic pollution load of the treatment works effluent at the final treatment structure in accordance with effluent guidelines in paragraph 11, below. Effluents shall meet all criteria expressed in Item b below.

b. All facilities for the treatment of sewage, industrial wastes, or other wastes shall provide for the following:

1) Substantially complete removal of settleable solids.

2) Removal of all floating debris, oil, grease, scum, or sludge solids.

3) Removal of color, odor, or turbidity to below obvious levels.

4) Removal of heavy metals, or of toxic and odor-producing substances in accordance with the levels of constituents and properties determined by the Board.

Section 11. Guidelines Regarding Range of Treatment.

a. Tertiary or other advance treatment or modifications of conventional treatment will be specified for all intermittent streams and small or low flow streams, and shall include effluent disinfection at least through the recreational season of April through October.

b. Permissive Treatment & Effluent Requirements Based on Average Strength Municipal Wastes

Type Treatment	BOD Reduction Percent	Effluent BOD mg/l	Effluent Suspended Solids mg/l	Type Facilities	Stream Dilution Requirements
Secondary	90	20	25	*Activated Sludge	2 to 1
*(or trickling filter & supplemental treatment)					
Tertiary	95	10	13	Secondary plus Supplemental	1 to 1
Advanced	98	4	5	Series Units	less than 1 to 1

Disinfection with up to 1 mg/l of chlorine residual in the effluent to reduce Coliform to 5,000 or less, where necessary, and reduce Fecal Coliform to 400 or less for primary contact waters or to 2,000 or less for secondary contact waters.

Bypass flows in excess of waste treatment works capacity shall be given primary treatment, and chlorination if necessary, in auxiliary facilities.

c. Within design limitations, operation shall be of such quality to obtain the best possible degree of treatment from all treatment works. Every effort must be made to eliminate all system bypasses and overflows, otherwise measures must be taken to provide treatment units such as lagoons, detention or holding basins, and chlorination. Installation of new combined sewers are prohibited. Existing combined sewer systems should be patrolled; overflow regulating devices shall be adjusted to convey the maximum practicable amount of combined flow to treatment facilities. Excess infiltration into the sewer system should be eliminated to keep dry weather flow within design limits of conduits and treatment works.

The basic design criteria used in developing the treatment units were taken from Technical Release 20-15 of the Illinois Sanitary Water Board. This publication outlined Sewage Treatment Design Criteria for sewage treatment works in the watersheds of State-Federal Impoundment Reservoirs such as at Shelbyville and Carlyle. Portions of the specific requirements of the Technical Release are quoted below:

1) All sewage treatment works shall be capable of providing at least 90-95 percent reduction of five-day BOD.

2) All sewage treatment works shall be provided with facilities for chlorination of effluent.

3) All sewage treatment works of mechanical type shall provide for By-Pass Holding Tanks or Lagoons.

    (a) Where concrete structures are proposed, volume required shall be such as to allow twenty-minute detention at 250 percent of dry weather flow (average design flow); chlorine shall be added at influent in sufficient quantity to insure residual in effluent.

    (b) Where lagoons are proposed, volume required shall be such as to allow five-day detention at dry weather flow. Depths up to eight feet (2.44 m) will be acceptable. Chlorination not required.

The Notice to Proceed was issued to the general contractor to begin construction of the facilities in Shelbyville April 29, 1969. On July 1, 1971, the facilities were completed and recommended for final acceptance. In the time between final design and final acceptance, there had been some substantial changes in the requirements for combined sewer overflow treatment. The design criteria and treatment requirements underwent further change and strengthening following final acceptance to evolve to the present rules and regulations.

On July 1, 1970, the Illinois "Environmental Protection Act" became effective establishing the Illinois Environmental Protection Agency, and the Illinois Pollution Control Board. On January 6 and March 7, 1972, the Board adopted new water pollution control regulations that provide a comprehensive package to replace former Sanitary Water Board Regulations. The new regulations established a set of General Standards for all water of the state as well as Effluent Standards for various discharges into the waters of the State. New regulations were also adopted for combined sewers and treatment plant bypasses, portions of which are outlined below.

> Rule 602 - Combined Sewers and Treatment Plant Bypasses
>
> (a) The installation of new combined sewers is prohibited, except where sufficient retention or treatment capacity is provided to ensure that no violation of the effluent standards.....occurs.
>
> (b) Excess infiltration into sewers shall be eliminated, and the maximum practicable flow shall be conveyed to treatment facilities. Overflows from sanitary sewers are expressly prohibited.
>
> (c) All combined sewer overflows and treatment plant bypasses shall be given sufficient treatment to prevent pollution or the violation of applicable water quality standards. Sufficient treatment shall consist of the following:
>
> (1) All dry weather flows, and the first flush of storm flows as determined by the Agency, shall meet the applicable effluent standards;
>
> (2) Additional flows, as determined by the Agency but not less than ten times the average dry weather flow for the design year, shall receive a minimum or primary treatment and disinfection with adequate retention time;
>
> (3) Additional treatment, through retention and return of excess flows to the treatment plant or otherwise, shall be provided when required to achieve compliance with water quality standards.

Compliance with Rule 602, paragraph (c) is required on or before December 31, 1975.

In developing a design for combined sewage pollution abatement in Shelbyville, the several different methods that were studied included:

1) Total separation of the combined sewer system.

2) Enlargement of the existing sewage treatment plant to handle the combined flows.

3) Separate treatment of storm flows and sewage flows.

A review of the literature shows that total separation of a combined sewer system is seldom economically practical, ranging in cost from $10,000 to $20,000 per acre. Based on preliminary engineering estimates, it was determined that total sewer separation in Shelbyville would cost in excess of $5 million and would be financially prohibitive to the City.

The treating of all the sewage flow of the combined sewer system, consisting of both dry weather and storm flows, in a conventional plant was also analyzed. It was apparent, though, that the hydraulic load in comparison to organic load on the plant during storm conditions would be detrimental to the biological treatment process. In addition, the cost of constructing a plant adequate in size would also be uneconomical for a community the size of Shelbyville. In order to provide adequate treatment of combined sewer flows, it would require the construction of a plant approximately six times the size of the plant necessary to treat only the dry weather flow.

The third treatment alternative, and the one selected for this project, consisted of partial separation of combined sewers and treatment of the remaining combined sewer overflows. Because of the natural drainage patterns in the city, treatment units were constructed at three locations, identified as the East Site, the Southeast Site, and the Southwest Site.

SECTION VI

POLLUTION ABATEMENT SYSTEM

SEWER SYSTEM IMPROVEMENTS

The original sewer system was installed prior to 1928, and before the recent improvements approximately 42% of the existing 14 miles (22.5 km) of sewers were combined sewers. To provide for efficient treatment of both sanitary and combined sewer overflow during wet weather, partial separation of the existing combined system was required. By separating the two flows as much as practical, several advantages could be realized. First, by removing sanitary flows from the larger combined sewers, the problem of solids deposition due to low velocity flows would be reduced. This would improve the system in several ways. Sewer maintenance to remove blockages and restrictions resulting from accumulated solids would be reduced, reductions in hydraulic capacity of the combined sewers due to grit accumulations would be arrested, and odor problems arising from sludge banks in the 4 x 5 foot (1.24 x 1.54 m) interceptor should be reduced or eliminated. Second, shock loads of BOD and suspended solids to the treatment plant resulting from the first flush effect of wet weather flow would be reduced. Third, dilution of sanitary sewage strength by storm water would be reduced, thus improving organic loading and treatment efficiency through the trickling filter during wet weather. Fourth, hydraulic overloading and potential sewer backups and flooding would be greatly curtailed by removing wet weather flows from under-sized sewers primarily designed for sanitary service. The completed improvements to the system included construction of three general types of sewers:

Sanitary Sewers - approximately 17,000 feet (5290 m) of 8-inch (20.4 cm) through 21-inch (53.5 cm) diameter extra strength vitrified clay sanitary sewer was installed to remove sanitary sewage from presently overloaded sanitary sewers and to intercept the dry weather flow from the large combined sewers. The sanitary sewers were designed using conventional design standards to maintain a minimum velocity of 2.0 ft/sec (61.2 cm/sec) and serve an ultimate population of 11,700 persons with an expected total flow of 1.17 MGD (4.4 MLD). Outlet for the sanitary sewers is to the existing sewage treatment plant.

Sanitary Relief Sewers - approximately 11,500 feet (3520 m) of 15-inch (38 cm) through 60-inch (154 cm) diameter reinforced concrete sanitary relief sewer was installed to intercept flow presently reaching sewers intended primarily for carrying sanitary sewage, and too small for handling storm flow. By-passes were also provided from the small combined sewers to the sanitary relief sewers to intercept storm flows from these small sewers. The sanitary relief sewers were designed to maintain a minimum velocity of 2.0 ft/sec (61.2 cm/sec) and handle a two year design storm. Outlet for the sanitary relief sewers is to the storm holding lagoon at the Southwest Site.

Combined Sewers - The existing combined sewer at the east side of Shelbyville was repaired and improved by adding about 2700 feet (823 m) of 12-inch (34 cm) through 21-inch (53.4 cm) reinforced concrete sewer pipe. Due to the steepness of the existing ground surface along the reach of this interceptor, it was possible to repair and improve the sewer so that a minimum velocity of 2.0 ft/sec (61.2 cm/sec) could be maintained for the expected average dry weather flow. The east side interceptor terminates at the East Site where all the dry weather flow is conveyed to treatment, and combined sewer overflow during wet weather periods is treated within the design limits of the system. Excess wet weather flow is chlorinated and bypassed to the Kaskaskia River.

With the completion of the sewer system improvements, all dry weather flow is conveyed to treatment before discharge to receiving waters, and all but about 12,000 gallons (45,300 l) per day is treated at the trickling filter plant. The small remainder is presently receiving lagoon treatment at the Southeast Site.

SOUTHEAST SITE

The topography and existing combined sewers at the southeast portion of the city is such that an area of about 44 acres drains directly to the Kaskaskia River. This drainage district includes about half of the central business district and a small railroad switch yard and is divided by natural ravine terminating in what used to be an intermittantly flowing creek extending about 300 feet (92.5 m) to the river. Dry weather flow is intercepted from a 36-inch (91.5 cm) combined sewer interceptor and conveyed to an existing lift station where it is pumped into the gravity flow system leading to the trickling filter plant. During wet weather, however, excess combined sewer flow bypasses the pump station and flows through a storm sewer terminating in a 48-inch (124 cm) cast iron outfall into the ravine. Since the volume of run-off from this district is much greater than the available capacity of the interceptor leading to the Southwest Site, and the cost of a new storm sewer and pump station would be prohibitive, only two alternatives remained. Complete sewer separation or combined sewer overflow treatment. Complete separation was ruled out since most of the area is either central business district or filled-in ravines with a difference in elevation of about 50 feet (15.4 m). Thus, the most feasible alternative was to provide treatment for the combined sewer overflow during wet weather, and continue to pump dry weather flows to the treatment plant with the existing pump station.

Since the combined sewer outfall already terminated in the ravine, a single cell lagoon was designed and constructed by building an earthen embankment across the lower reaches of the ravine.

The layout of the single cell lagoon located at the Southeast Site is shown in Figure 6. The lagoon serves an area of 44 acres (178,000 sq. m.) with an equivalent population of 615 persons. The principal design criteria of the single cell lagoon is as follows:

(a) Equivalent dry weather flow	61,500 gpd (233,000 LPD)
(b) Lagoon capacity - 620% of 5 days equivalent dry weather flow	255,600 cf (7,250 cm) at 8 ft. (2.44 m) depth
(c) Depth range	0 to 8 ft. (0 - 2.44 m)
(d) Surface Area @ 8 ft.(2.44 m)	1 acre (approx.) (4,047 sq.m.)
(e) Equivalent storm contained (assuming 50% runoff)	in excess of 10 year storm of 1 hour duration (1.95 in/hr) (4.95 cm/hr)

Although most of the dry weather flow is diverted to the existing raw sewage lift station, there is always about 12,000 gallons (45,000 l) per day of dry weather flow into the lagoon. Part of this dry weather flow originates from several dwelling units in the immediate vicinity and the remaining flow is from undetermined origins. This undiverted

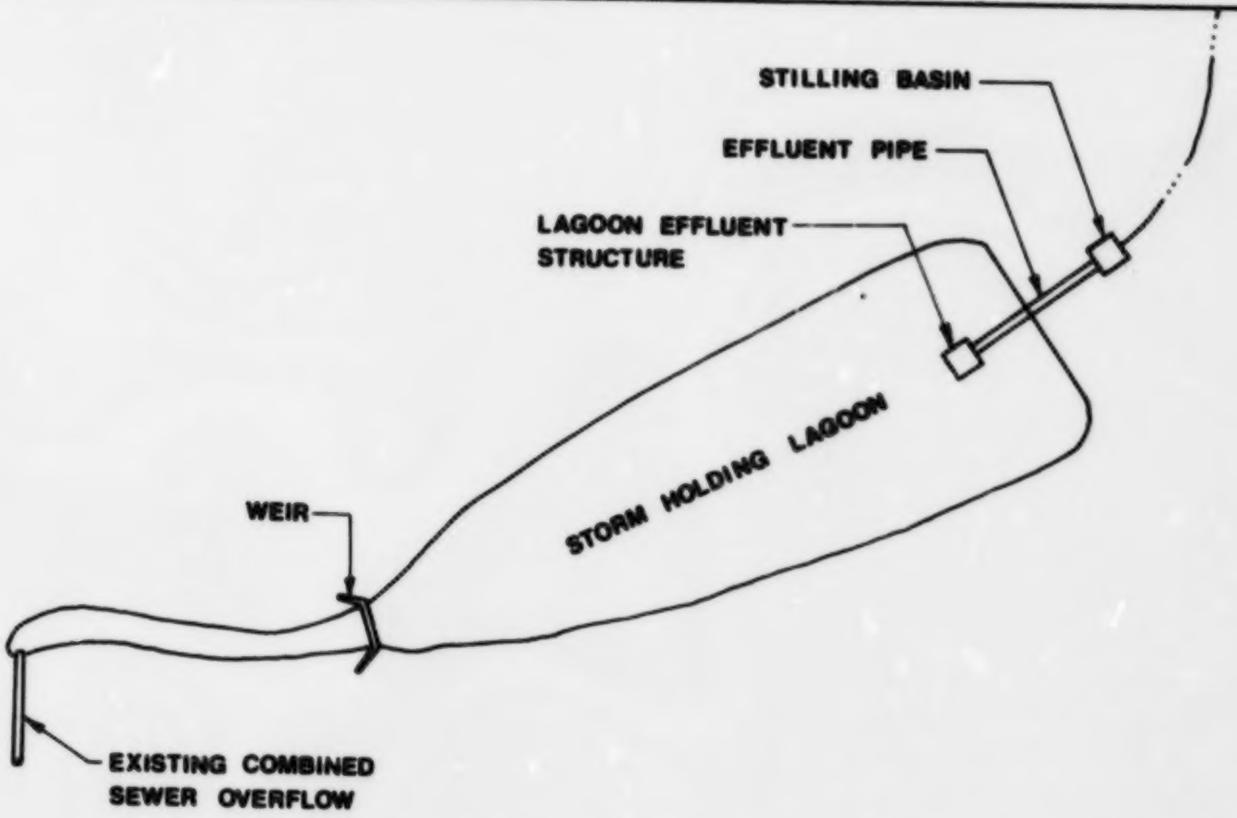

STILLING BASIN
EFFLUENT PIPE
LAGOON EFFLUENT STRUCTURE
STORM HOLDING LAGOON
WEIR
EXISTING COMBINED SEWER OVERFLOW

Figure 6. Southeast Site Treatment System

flow is believed to be, in part, due to infiltration, unidentified sewer connections, and a poorly maintained dry weather bypass to the existing sewage lift station. When sewer repairs are made and the system is cleaned, most, if not all of this flow will be eliminated.

The lagoon effluent structure has two twelve-inch (30.5 cm) square sluice gates mounted in the structure wall. One gate is located at an elevation corresponding to the lagoon bottom and the second gate is mounted two feet (.612 m) higher. The lower gate is normally closed, and the upper one is set to drain a full lagoon to the two foot (.612 m) level in five days.

The effluent structure is designed as a 38-foot (11.6 m) broad-crested weir in the shape of a rectangle to provide for overflow from a 25 year storm of 1 hour duration (i.e. 62 MGD). This would allow two feet (.612 m) of freeboard at the effluent embankment.

Lagoon effluent passes through the effluent pipe into an energy dissipating stilling basin to minimize erosion at the headwaters of the receiving stream.

Normal lagoon operation is a flow-through pattern, and during dry weather the average water depth is 24 inches (61.2 cm). During wet weather, the lagoon discharge increases as the depth of water over the effluent sluice gate increases. Normally the gate requires no operator attention and is set to an opening such that the lagoon will drain from the 8 foot (2.44 m) depth (full) to the 2 foot (.61 m) depth in about 5 days.

EAST SITE

The conditions on the northeast side of Shelbyville are similar in many respects to those described for the southeast side. Two large, but shallower, ravines converge at a location along the river. The drainage district is slightly smaller than the Southeast Site, and prior to the sewer system improvements was served by a deteriorating combined sewage interceptor with an outfall to the Kaskaskia River. The same arguments against pumping the combined sewage flow to the Southwest Site, or providing separate sanitary and storm sewers that applied to the southeast drainage district were also appliable to the northeast district. Since the slope of the interceptor was great enough to provide a minimum 2.0 ft/sec (61.2 cm/sec) velocity at dry weather flows, a raw sewage lift station was located at the lower reach of the interceptor to pump dry weather flows into the gravity system leading to the treatment plant. To provide treatment for the combined sewage flows during wet weather, a storm water holding and treatment tank was constructed to intercept flows exceeding the pump station capacity and provide primary treatment and chlorination.

The facilities provided for handling dry weather and wet weather flows at the east side of Shelbyville are shown diagramatically in Figure 7. The raw sewage pump station, designed to handle all dry weather flows, was sized by conventional standards to handle 2.5 times the design flow. The principal design criteria are as follows:

(a) Present Population Served	800 persons
(b) Design Population	1,000 persons
(c) Average Design Flow	100,000 gpd-70 gpm (378,500 LPD)(266 LPM)
(d) Capacity each Pump (2.5 x Average Design Flow)	175 gpm (660 LPM)

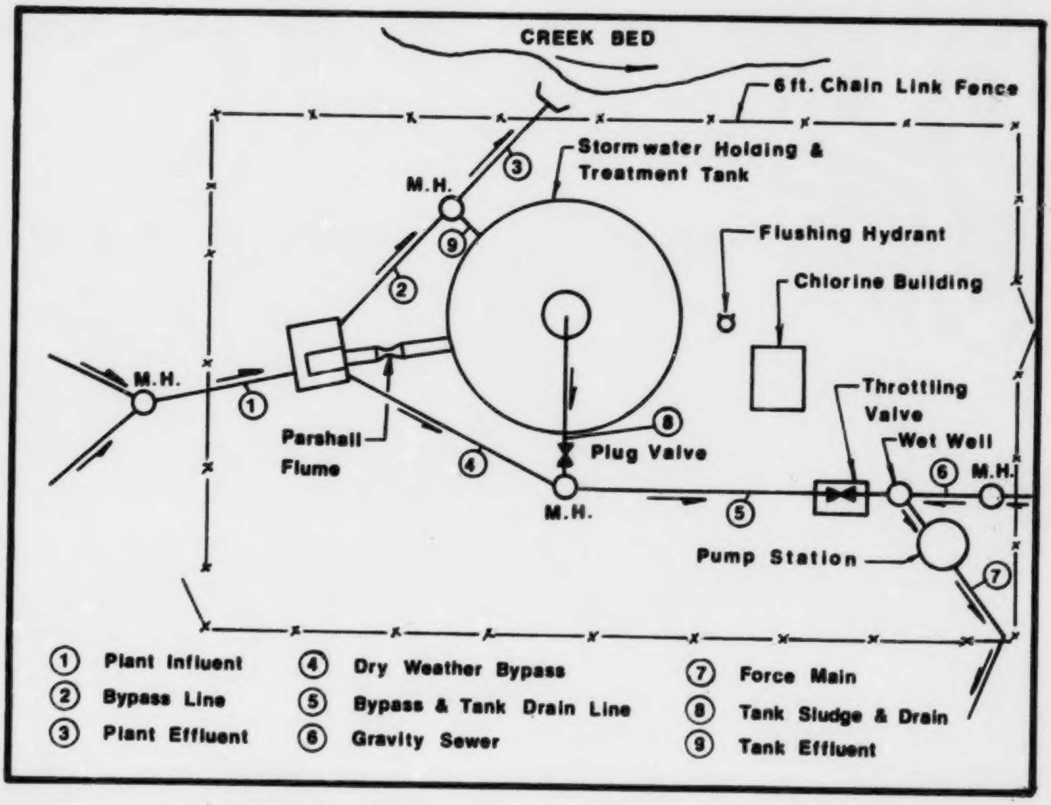

CREEK BED
6 ft. Chain Link Fence
Stormwater Holding & Treatment Tank
Flushing Hydrant
Chlorine Building
M.H.
M.H.
Parshall Flume
Plug Valve
M.H.
Throttling Valve
Wet Well
M.H.
Pump Station
① ② ③ ④ ⑤ ⑥ ⑦ ⑧ ⑨
① Plant Influent
② Bypass Line
③ Plant Effluent
④ Dry Weather Bypass
⑤ Bypass & Tank Drain Line
⑥ Gravity Sewer
⑦ Force Main
⑧ Tank Sludge & Drain
⑨ Tank Effluent

Figure 7. East Site Treatment System

The storm water holding and treatment tank and comminutor are sized to handle 600% of the average dry weather design flow as required by the Illinois Sanitary Water Board regulations that were in effect at the time of design. The principal design criteria of the treatment tank system are as follows:

(a) Design Flow	600,000 gpd (2,270,000 LPD)
(b) Maximum Flow	1,600,000 gpd (6,050,000 LPD)
(c) Detention Time at: 0.60 MGD (2.27 MLD)	125 min.
1.6 MGD (6.05 MLD)	50 min.
(d) Surface Settling Rate at 0.60 MGD (2.27 MLD)	850 gpd/sq.ft. (34,800 LPD/sq.m)
(e) Weir Overflow Rate at 0.60 MGD (2.27 MLD)	6,700 gpd/lin.ft. (83,000 LPD/lin.m.)

A 500 pound (227 kg)-per-day chlorinator with flow proportional control automatically feeds chlorine solution when flows are entering the treatment tank. Chlorine dose range can be set in excess of 25 mg/l.

Normal dry weather flows pass directly to the pump station wet well. A modulating valve ahead of the pump station regulates the level in the wet well to limit total pumpage during wet weather.

Wet weather flows exceeding the pump station capacity are by-passed through a Parshall flume and comminutor into the storm treatment tank. As soon as flow is detected in the Parshall flume, the chlorinator, comminutor, and sludge scraper mechanism in the tank automatically start.

Combined sewage flows in excess of the maximum design flow to the treatment tank are bypassed around the treatment tank and flow directly to the receiving stream. The excess flow bypass structure consists of a sharp crested rectangular weir with an equivalent length of twelve feet. This weir is set so that at the maximum hydraulic capacity of the influent sewer the total flow to the treatment tank is 1.6 MGD (6.05 MLD). The chlorine solution diffuser is located so that all bypassed flow, as well as that entering the tank, is chlorinated.

After each overflow event in which flows enter the tank, the tank sludge line must be manually opened by the plant operator to drain the tank and accumulated sludge into the pump station wet well where it is pumped into the gravity sewer system leading to the treatment plant.

SOUTHWEST SITE

The existing trickling filter plant and 4 x 5 foot (1.24 x 1.54 m) combined sewage interceptor outfall are located at the southwest corner of the city limits of Shelbyville. With the new sewer improvements, the entire dry weather flow can now be conveyed to the headworks of the plant through a new 21-inch (54.3 cm) sanitary sewer.

A bypass structure is located immediately ahead of the plant to relieve hydraulic overloads resulting from wet weather combined sewer flows. The bypass channels excess flow in the new sanitary sewer line back to the 4 x 5 foot (1.24 x 1.54 m) combined sewage interceptor.

No improvements were made to the existing plant under this project. Despite the old equipment and need for some additional maintenance, the plant produces a dry weather effluent with 70 to 80% BOD and suspended solids reduction.

To provide treatment for combined sewer overflow during wet weather, as well as tertiary treatment for the trickling filter plant effluent, a storm holding lagoon and a two-cell facultative lagoon system were constructed.

The general layout of the treatment facility is shown diagramatically in Figure 8.

The storm holding lagoon receives all combined sewer overflow from the 4 x 5 foot (1.24 x 1.54 m) interceptor and bypassed flows from the new sanitary sewer during wet weather. The lagoon was sized to contain the expected run-off from a 10 year storm of 1 hour duration and is capable of handling the maximum hydraulic load from the 4 x 5 foot (1.24 x 1.54 m) interceptor sewer. The principal design criteria are as follows:

(a) Capacity - runoff from a 10-year storm of 1-hour duration assuming 50% runoff	535,000 cf (15,200 cu.m)
(b) Depth Range	0-8 ft. (0-2.44 m)

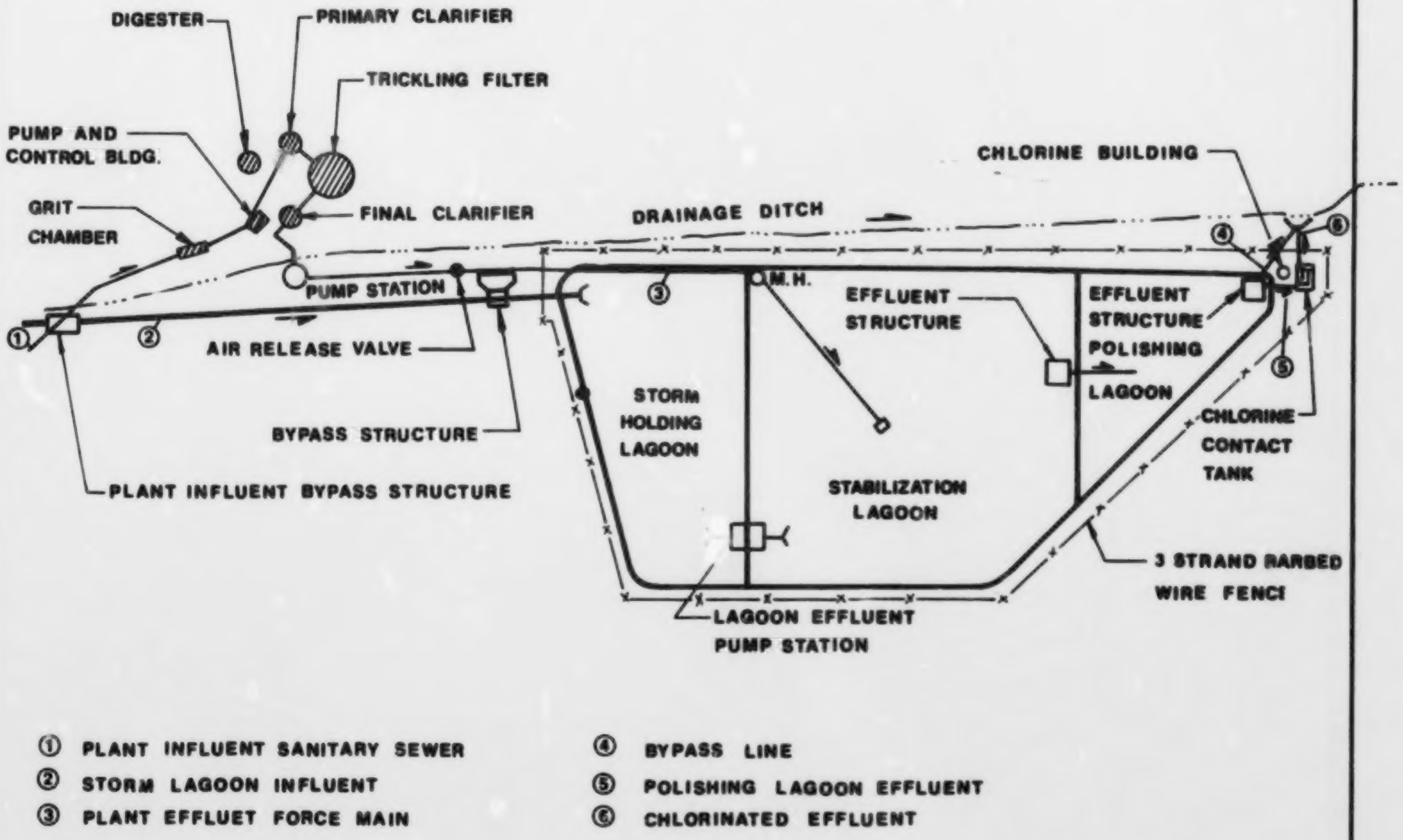

DIGESTER
PRIMARY CLARIFIER
TRICKLING FILTER
PUMP AND CONTROL BLDG.
GRIT CHAMBER
FINAL CLARIFIER
DRAINAGE DITCH
CHLORINE BUILDING
PUMP STATION
M.H.
EFFLUENT STRUCTURE
EFFLUENT STRUCTURE POLISHING LAGOON
AIR RELEASE VALVE
BYPASS STRUCTURE
PLANT INFLUENT BYPASS STRUCTURE
STORM HOLDING LAGOON
STABILIZATION LAGOON
CHLORINE CONTACT TANK
3 STRAND RARBED WIRE FENCI
LAGOON EFFLUENT PUMP STATION
① PLANT INFLUENT SANITARY SEWER
② STORM LAGOON INFLUENT
③ PLANT EFFLUET FORCE MAIN
④ BYPASS LINE
⑤ POLISHING LAGOON EFFLUENT
⑥ CHLORINATED EFFLUENT

Figure 8. Southwest Site Treatment System

(c) Surface Area at 8 ft. (2.44 m)	3.90 acres (1580 sq.m.)
(d) Maximum Flow	110 MGD (415 MLD)
(e) Area served	450 acres(est.) (1,820,000 sq.m.)

A bypass structure is provided at the influent of the storm holding lagoon to divert all flow into the receiving stream when the lagoon is full. A 28 foot (8.55 m) sharp crested rectangular weir is located at an elevation corresponding to the high water level of the full lagoon (8 ft.)(2.44 m). At a maximum hydraulic flow in the influent sewer of 110 MGD (415 MLD), the water crest over the weir when the lagoon is full will be no greater than 1.5 feet (.457 m).

Effluent from the storm holding lagoon is pumped into the stabilization lagoon by a lagoon effluent pump station consisting of two 500 gpm (1890 LPM) pumps operating together automatically based on storm holding lagoon levels. Together the pumps can empty the lagoon in about 5 days.

The effluent from the existing trickling filter plant is pumped from the plant outfall by a pump station to the stabilization lagoon. In addition, during wet weather and for a period following wet weather, effluent from the storm holding lagoon can be pumped to the stabilization lagoon. The stabilization lagoon has been designed and sized to provide tertiary treatment for the plant effluent. By using a conservative loading, it was determined that the additional loads resulting from storm holding lagoon effluent would not cause any serious organic overload. The following design criteria were used to size the stabilization lagoon.

(a) Design Population Equivalent (1987 design year)	8000 PE
(b) BOD/Capita to Plant	0.17 lbs/day (.077 kg/day)
(c) Treatment Plant Efficiency	80%
(d) Total BOD to Lagoon	272 lbs/day (124 kg/day)
(e) Surface Area of Lagoon	10.8 Acres (43,800 sq.m.) @ 4 ft.depth (1.22 m)

(f) Dry Weather Lagoon Loading	25.6 lbs BOD/ac/day (.0028 $\frac{\text{kg BOD}}{\text{sq m/day}}$)
(g) Depth Range	0-6 ft. (0-1.83 m)

The effluent from the stabilization lagoon flows by gravity through an effluent structure into the polishing lagoon. Assuming a 75% BOD removal efficiency in the stabilization lagoon, the second lagoon is designed for approximately the same loading rate as the first. Principal design criteria are:

(a) Total BOD to Polishing Lagoon	54.5 lbs/day (24.8 kg/day)
(b) Surface Area of Polishing Lagoon	2.1 acres (8500 sq m) @ 4 ft.depth (1.22 m)
(c) Dry weather lagoon loading	25.9 lbs BOD/ac/day (.0029 $\frac{\text{kg BOD}}{\text{sq m/day}}$)
(d) Depth Range	0-6 ft. (0-1.83 m)

Chlorination of the final lagoon effluent is by a 500 lb/day (227 kg/day) solution feed type chlorinator paced by flow signals from the effluent Parshall flume. At the maximum flow of 3.46 MGD (13.1 MLD) (plant effluent and storm holding lagoon effluent) maximum chlorine dose is in excess of 17 mg/l.

Assuming average BOD removal rates through the trickling filter of 85%, and 75% removal across each lagoon, the anticipated total dry weather treatment efficiency would be in excess of 90%.

The operation of the treatment units at the Southwest Site is automatic under normal conditions. During dry weather, plant effluent is pumped to the stabilization lagoon where it begins a continuous flow-through pattern leading to discharge from the chlorine contact tank. Lagoon levels are controlled by stop logs at the respective effluent structures, and generally maintained at a constant level year around.

The storm holding lagoon is maintained at a dry weather water depth of 18 to 24 inches (45.7 to 61.2 cm) to discourage aquatic weed growth and maintain some dilution capacity. During wet weather, the effluent pumps automatically start when the depth attains six feet and continue pumping until the 18-24 inch (45.7-61.2 cm) level is again reached. When the run-off from a storm is not great enough to fill the lagoon to the 6 foot (1.83 m) depth, the operator can manually start the pumps after 3 to 5 days, depending on probabilities of further precipitation.

SECTION VII

EVALUATION OF TREATMENT UNIT OPERATION

SAMPLING

Since the treatment units constructed for the City of Shelbyville have both dry weather and wet weather functions, sampling programs were devised to show the degree of treatment during both dry weather and wet weather periods. The sampling programs included both flow measurement through the respective units and chemical and physical analyses of influent flows. Tables 6 and 7 outline the sampling locations and methods for each unit during the dry and wet weather periods.

All flow measurements were made using recording instruments to indicate the head of water at the primary element. Head readings for all flow data were converted to units of flow in the laboratory except for flume readings which were indicated directly in flow units. The existing Parshall flume in the treatment plant influent channel was not found to be sufficiently accurate, so a correlation graph was prepared using a temporary sharp crested weir ahead of the flume. The raw influent flow data was then corrected according to the correlation graph.

Sampling intervals varied according to the expected rate of change of flow and analytical parameters. For dry weather sampling, the automatic samplers were set to collect one sample each hour beginning about noon each day. Samples were then composited in groups representing 24 hours, weighted according to flow, iced down, and delivered to the laboratory for analysis. Dry weather flow into the southeast lagoon was essentially constant, so these samples were composited on an equal volume basis. Grab samples were collected daily about noon and were not composited. For the composite samples, dry weather data is reported for the 24 hour period ending about noon on the date indicated.

Wet weather sampling was conducted at more frequent intervals to better reflect the changing characteristics of the flow. Automatic samplers were set to collect one sample every 15 minutes and were started before any significant wet weather flows occurred. Grab samples were taken before rainfall occurred and at 3 to 4 hour intervals until the run-off decreased significantly. When all 15 minute samples were collected (24), a second sampler was installed to continue collecting samples at either 15 minute or 60 minute intervals, depending on the progress and intensity of the storm. Both

TABLE 6

DRY WEATHER SAMPLING PROGRAM

LOCATION	INFLUENT FLOW	EFFLUENT FLOW	INFLUENT SAMPLE METHOD	EFFLUENT SAMPLE METHOD
EAST SITE	None	None	None	None
SOUTHEAST SITE	*14 ft. (4.27 m) sharp crest rectangular weir	90° V-notch weir	Automatic	Grab
SOUTHWEST SITE Treatment Plant	Existing 12 in.(30.5cm) Parshall flume	None	Automatic	Automatic
Storm Holding Lagoon	None	None	None	None
Stabilization Lagoon	None	None	None(same as plant effluent)	Grab
Polishing Lagoon	None	None	None(same as Stabilization effluent)	Grab
Final Effluent (Chlorination)	None	9 in. (22.8 cm) Parshall flume	None(same as Polishing Lagoon Effluent)	Grab

*Two small 90° V-notches cut in the 14 foot (4.27 m) rectangular weir plate for low flow measurements.

TABLE 7

WET WEATHER SAMPLING PROGRAM

LOCATION	INFLUENT FLOW	EFFLUENT FLOW	INFLUENT SAMPLE METHOD	EFFLUENT SAMPLE METHOD
EAST SITE	6 in.(15.2cm) Parshall flume	None	Automatic	Automatic
SOUTHEAST SITE Treatment Plant	12 in. (30.5 cm) Parshall flume	None	Automatic	Grab
Storm Holding Lagoon	72 in. sharp (1.83 m) crested rectangular weir	Pump run time	Automatic	Grab
Stabilization Lagoon	None	None	None(same as plant effluent & storm lagoon effluent)	Grab
Polishing Lagoon	None	None	None(same as Stab. lagoon effluent)	Grab
Final Effluent	None	9 in. (22.8 cm) Parshall flume	None(same as polishing lagoon effluent)	Grab

automatic sampling and grab sampling were continued until the run-off had reached minimal flows. Automatically collected samples were composited according to flows to obtain a series of samples from the start to finish of the run-off flow. Grab samples were either analyzed individually or composited to similarly provide data at various intervals for each storm event. Lagoon effluents were sampled for several days following each storm to obtain data on lagoon efficiency after periods of high loading.

After collection, samples were refrigerated and transported in ice chests to the laboratory for analysis. No preservatives were added to any sample, and all analyses were performed at Purdue University according to accepted Standard Methods techniques.

DRY WEATHER DATA

An extensive dry weather sampling program was carried out at the Southeast and Southwest Sites over a six-day period from 19-July, 1971 through 24-July, 1971. The purpose of this initial program was to collect enough data to accurately characterize the average dry weather flow and to demonstrate the degree of treatment provided by the various units.

Flow data for the Southeast Site is incomplete because a severe storm earlier in the month destroyed the measuring weir at the influent to the lagoon. This storm, which deposited 4.02 inches in six hours, was equivalent to a 25 year storm of six hours duration, and delayed the start of dry weather sampling. Because flow into the lagoon was typical of previous dry weather periods, and did not vary throughout the day, it was decided to proceed with the sampling program before the weir was completely rebuilt.

Flow data for the Southwest Site was also incomplete. It was discovered at the completion of the sampling program that the plant flow measuring equipment was out of caligration. Although flow charts were available, it was determined that they were inaccurate, and could not be used to obtain reliable flow data for this report. In spite of these shortcomings, it was felt that the dry weather data did provide an accurate measurement of unit operation and was of sufficient value to be retained.

A second dry weather sampling program was conducted for a 24-hour period beginning 1-June, 1972. The purpose of this second evaluation of dry weather operation was both to confirm the results obtained the first time and to provide data on the flows and loadings of the treatment units. This second sampling period was only for evaluation of BOD, suspended solids, and fecal coliform.

The results of the two dry weather sampling programs are tabulated in Tables 8 through 13. No dry weather sampling was performed at the East Site because operation is similar to conventional sewage lift stations. The entire dry weather flow is intercepted at the lift station wet well, and pumped through the force main into the gravity flow sewer system leading to the existing treatment plant. No dry weather flow is bypassed nor treated at the East Site.

DRY WEATHER PERFORMANCE

Southeast Site - The expected performance of single-cell lagoons as a domestic waste treatment unit has been thoroughly characterized in the literature and will not be reviewed in this report. The data collected during dry weather operation at the Southeast Site is similar to the expected results for summer operation of facultative lagoon in which no algae removal is practiced.

The influent flow was measured by two 90 degree V-notches cut in the 14 foot (4.26 m) rectangular weir, and was irregularly variable, ranging between about 13 gpm and 30 gpm (.83 and 1.89 l/sec) in a seemingly random fashion. Changes in flow rates were abrupt rather than gradual, suggesting either a periodic discharge such as a process water, or a restriction in flow such as might be caused by random blockage of a dry weather flow diversion structure.

Subsequent investigation of the source of the dry weather flow using tracer dye revealed that no flow was entering the lagoon from either the pumped discharge at the East Site, or through the diversion structure at the existing lift station at the southeast side of town. Flow was entering the lagoon, however, through a portion of the combined sewer system that was rebuilt by the City after the completion of the abatement project, and further investigation revealed a bypass line to the existing lift station that was non-functioning due to blockage by silt and debris. Attempts to restore this bypass line to permanent service were not ultimately successful, and engineering services will probably be required to determine a solution to this problem.

Infiltration is recognized in portions of the combined sewers leading to the lagoon, but there is no reliable evidence to suggest any serious infiltration into the lagoon itself. The flow data for the 24 hour dry weather period does show a net increase in effluent volume with respect to influent volume, however, various factors can contribute to this apparent error. The accuracy of the lagoon stage recorder is only ±0.1 foot (3.05 cm). A change in the water surface elevation by this amount is a net change in volume of about 20,000 gallons (76,000 l), so in measuring small rates of flow, appreciable imbalance can be observed from small changes in water surface elevation and such factors as wind set-up. A study of several months of continuous flow data does not suggest any gross infiltration into the lagoon.

In the absence of any average net imbalance in flow into and out of the lagoon, the simple percent changes of analytical parameters shown in Table 8 for the dry weather data are valid. Since accurate flow data was available for the 24-hour dry weather period, the percent removals are expressed in terms of the actual loads applied to the lagoon and receiving stream.

The influent BOD and suspended solids values are less than what would be expected for domestic sewage, suggesting that either infiltration of ground water, or process or cooling water, may be diluting the raw sewage. Neither of these possibilities have been verified in the field.

During all dry weather sampling, the lagoon was at the two foot (61 cm) depth, providing a surface area of approximately 0.6 acres (2400 sq m) and a mean detention time of about 6 days. The average BOD loading was 31.7 pounds per acre per day (.112 g/sq m/day).

The removal efficiency of the single cell lagoon, based on the 24-hour data, is somewhat less than the usually quoted 50day BOD reduction of 75%. The computed reductions, determined by the mass difference between influent and effluent, were 47.5% for BOD and 64.5% for suspended solids. Soluble BOD values were not determined.

Effluent coliform was reduced significantly across the lagoon. Total coliform decreased an average of 86% during the 6-day dry weather period and fecal coliform averaged 82% reduction during the first sampling period, and a 97% reduction during the 24-hour period. These natural removal rates could, of course, be increased to nearly 100% with the addition of effluent chlorination.

The nutrient requirements for nitrogen and phosphorous are amply satisfied in the influent flow, and the phosphate, ammonia, and nitrate levels are significantly reduced across

TABLE 8

SOUTHEAST LAGOON DRY WEATHER OPERATION

Parameter	6-Day Dry Weather Operation Influent	Effluent	Net Change
BOD	29.4 mg/l	15.7 mg/l	47%
COD	70.8 mg/l	52.0 mg/l	27%
TS	944 mg/l	734 mg/l	22%
TVS	330 mg/l	232 mg/l	30%
SS	72 mg/l	31 mg/l	57%
VSS	24.8 mg/l	7.3 mg/l	71%
PO_4	7.8 mg/l	4.7 mg/l	40%
NH_3-N	7.7 mg/l	2.3 mg/l	70%
NO_2-N	.084 mg/l	.082 mg/l	2%
NO_3-N	.20 mg/l	.04 mg/l	80%
N-organic	2.16 mg/l	2.16 mg/l	5%
Spec. Cond.	1133 u /cm	942 u /cm	17%
Total Coliform	197×10^4/100 ml	27×10^4/100 ml	86%
Fecal coliform	22×10^4/100 ml	4×10^4/100 ml	82%
	24-Hour Dry Weather Operation		
FLOW	52,600 gal	70,900 gal	--
BOD_5	44 mg/l	17 mg/l	47.5%
Suspended Solids	72 mg/l	18 mg/l	64.5%

Lagoon Loading at 2 ft (61 cm) depth, 0.6 acre (2430 sq m) surface area = 31.7 # BOD per acre per day (3.6 g/sq m/day) the lagoon. Slight reductions (5%) in the average level of organically bound nitrogen is also indicated by the data.

Southwest Site - The six-day dry weather data indicates that the strength of the influent raw sewage increased significantly during the sampling period, probably due to a progressive decrease in ground-water infiltration. The ground-water table for the area has not been studied in detail, but results from soil borings indicate that in some locations the water table may only lie two feet (0.61 m) below the surface. Total rainfall during the two weeks preceeding the sampling period totaled eight inches (20 cm) and probably elevated the ground water level in some portions of the system, increasing the extent of infiltration. Since dry weather flows are intercepted at various locations in the combined sewers, and because the older combined sewers are of brick construction, there was undoubtedly appreciable dilution of the sanitary flows by infiltrating grount water.

The magnitude of the raw sewage influent flows during the six-day sampling period is not accurately known. Plant flow measuring equipment was later found to be out of calibration, and although relative flow rates were recorded, it was not possible to calculate the true flows. It was later learned that plant influent flows were arbitratily regulated with the influent sluice gate by plant personnel, causing some bypassing of the peak flows to the storm holding lagoon. This was not felt to be a serious fault, however, since the bypassed flow had the same characteristics as that taken through the plant, and the amounts were not great. This bypassed flow did not enter the dry weather treatment process, but was retained for later discharge into the tertiary lagoon system.

Flow data for the 24-hour dry weather period was accurately taken, and no bypassing was permitted. A study of final effluent flow records for a 10 month period indicate that the average dry weather flow through the plant is about 500,000 gallons per day (1,890,000 l/day). During wet weather periods, infiltration due to a higher ground water level, as well as a higher soil moisture content, increase the average influent flows by as much as 100%, dependent on the amount of rainfall.

The extent of ground water infiltration into the sewer system could be reduced if a comprehensive sewer maintenance program were undertaken, but until dry weather flow is completely eliminated from all the old brick combined sewers a certain amount of infiltration will persist.

Influent BOD levels varied considerably during the six-day dry weather sampling period, but the variations in loading did not appear to have much effect on the removals, suggesting that the plant could effectively handle the diluted influent without serious degradation in performance. Suspended solids removals through the treatment plant averaged 89% during the six-day sampling period, and both total and fecal coliform had greater than 95% removals. (Table 9)

The twenty-four hour dry weather data did not show the same degree of treatment measured during the six-day period. Influent BOD was about the same as previously measured, but removal was not nearly as high. Influent suspended solids averaged less than the previous results, and again removals were not as good. Fecal coliform was determined only in the treatment plant effluent, and the result was about two orders of magnitude greater than the average 6-day value.

The reason for the apparent poor treatment through the trickling filter plant during the 24-hour dry weather period is not clear, but several possibilities are suggested. One explanation is that the trickling filter was in the process of sloughing off solids, and the results represent the low values in the average treatment provided. A second possibility is that the trickling filter was in the process of recovering from a shock load and removal efficiency was low. This second explanation is the more likely of the two. The plant grit chamber was cleaned infrequently, and much of the gritty material is removed in the primary tank. Primary sludge is pumped to the anaerobic digester where it accumulates and decomposes. The digester is not carefully controlled according to a regular schedule. Two or three times a week the operator releases supernatant back to the primary tank in irregular quantities. This supernatant, which is probably very high in BOD and suspended solids, would be sufficient to retard the treatment process and subsequent effluent quality.

The tertiary lagoon system (2-cells) and chlorination facility provided further treatment to the plant effluent with an average reduction in all plant effluent parameters. The stabilization lagoon was operated at an average depth of 57 inches (1.45 m) and surface area of 10.9 acres (44,000 sq m), and the polishing was at 44 inches (1.12 m) depth and surface area of 2.0 acres (8,100 sq m).

During the six-day dry weather sampling period, where plant effluent BOD and suspended solids values averaged 13.5 mg/l and 20 mg/l, respectively, the measured BOD and suspended solids showed an increase across the stabilization lagoon

TABLE 9
SOUTHWEST SITE DRY WEATHER OPERATION
6-Day Dry Weather Period

Parameter	Raw Influent	Plant Effluent	Stabilization Lagoon Effluent	Polishing Lagoon Effluent	Final Effluent	% Removal
BOD	79 mg/1	135 mg/1	14.5 mg/1	12 mg/1	5 mg/1	94%
COD	130	47	50	45	42	77%
TS	830	678	488	470	505	39%
TVS	351	245	173	164	212	40%
SS	137	20	40	24	16	91%
VSS	135	19.0	14.7	13.7	7.3	95%
PO_4	20.0	14.1	6.2	4.4	3.8	81%
NH_3-N	8.4	1.6	0.8	0.4	0.2	98%
NO_2-N	0.03	0.18	0.12	0.06	0.00	100%
NO_3-N	0.08	3.47	0.36	0.02	0.20	-150%
Org-N	6.0	2.05	2.22	2.24	2.04	66%
Spec.Cond.	929 u /cm	892 u /cm	612 u /cm	586 u /cm	612 u /cm	34%
Total Coliform	$475x10^4$/100ml	$12x10^4$/100ml	$1.12x10^4$/100ml	$0.4x10^4$/100ml	0	100%
Fecal Coliform	$46x10^4$/100ml	$1.9x10^4$/100ml	$0.66x10^4$/100ml	$0.06x10^4$/100ml	0	100%

Negative % removals indicate a net increase from influent to effluent.

due to the prolific growth of algae. However, the nutrient parameters all showed a significant decrease:

	From	To
Total Phosphorous-PO_4	14.1 mg/l	6.2 mg/l
Ammonia Nitrogen	1.6 mg/l	0.8 mg/l
Nitrite Nitrogen	0.18 mg/l	0.12 mg/l
Nitrate Nitrogen	3.5 mg/l	0.4 mg/l
Organic Nitrogen	2.0 mg/l	2.2 mg/l

Organically bound nitrogen increased across the stabilization lagoon by about 8%, as would be expected with the metabolic activity in the lagoon.

BOD and suspended solids levels were reduced across the polishing lagoon, and further reduction in nutrient levels were observed. The increase in organically bount nitrogen was much less than in the preceeding lagoon, indicating a lower rate of metabolic activity.

Chlorination of the final effluent further reduced the BOD and suspended solids, as well as the remaining phosphorous, ammonia, and nitrites. The organically bound nitrogen was oxidized by the chlorine to nitrate.

The net results of the tertiary treatment system during the six-day dry weather sampling period were significant reduction in BOD, and nutrient levels and moderate reduction of suspended solids. Naturally, the suspended solids and BOD removals would be greater if some method of algae removal were used before final discharge. Total and fecal coliform levels showed an average decrease through the lagoon system, with no coliform organisms detected in the final chlorinated effluent. A summary of the average 6-day dry weather data is shown in Table 9.

The 24-hour dry weather performance of the tertiary lagoon system was not as good as the previous 6-day period. Unfiltered BOD removals for the 24-hour period averaged only 46% while suspended solids showed a net increase of 26%. Examination of the data for each lagoon indicates that the major reduction in BOD and increase in suspended solids occurred in the stabilization lagoon with no further improvement in the polishing lagoon and only slight improvement with chlorination. (Table 10)

The reason for the lower treatment efficiency through the tertiary treatment system is not immediately clear. The BOD loading to the stabilization lagoon, based on a flow of

TABLE 10
24-HOUR DRY WEATHER OPERATION

Date	Time	Bod mg/l	Suspended Solids mg/l	Fecal* Coliform MPN/100ml x 10^{-4}	Sample Type Composite or Grab
PLANT INFLUENT					
6-1-72	12:00PM- 5:00PM	141	95		C
	6:00PM-12:41AM	129	78		C
6-2-72	1:41AM- 3:41AM	60	60		C
	4:41AM-11:41AM	111	126	--	C
Average		110.2	89.8		
PLANT EFFLUENT					
6-1-72	11:51AM- 5:07PM	42	45		C
	6:01PM-11:01PM	43	45		C
6-2-72	12:07AM- 5:01AM	56	59	150	C
	6:51AM-11:51AM	26	27		C
Average		41.1	44.0	150	
STABILIZATION LAGOON EFFLUENT					
6-1-72	1:35 PM	23	70	9.3	G
	7:00 PM	20	48		G
6-2-72	1:17 AM	21	68		G
	6:37 AM	29	72		G
	12:06 PM	23	69		G
Average		23.2	66.4	9.3	
POLISHING LAGOON EFFLUENT					
6-1-72	1:41 PM	20	60		G
	6:56 PM	27	60		G
6-2-72	1:13 AM	22	70	4.3	G
	6:20 AM	37	69		G
	11:57 AM	23	53		G
Average		25.8	62.4	4.3	
FINAL EFFLUENT					
6-1-72	1:43 PM	20	42		G
	6:49 PM	16	48		G
6-2-72	1:10 AM	27	68	0.0	G
	6:25 AM	28	64		G
	11:55 AM	19	55		G
Average		22.0	55.4	0	

*Composite Sample

Average Removals:	Plant	Stabilization	Polishing	Chlorination	Total
BOD	62.1%	43.5%	-11.2%	14.7%	80.0%
Suspended Solids	51.0%	-50.9%	6.0%	11.2%	38.4%
Fecal Coliform	--	93.8%	53.8%	100.0%	100.0%

520,000 gallons (1,960,000 l) per day, is 100 pounds (45.4 kg) or about 9.2 pounds of BOD per acre per day (1.03g/sq m). For the polishing lagoon, the loading computes to about 111 pounds (50.4 kg) or 55 pounds of BOD per acre per day (6.16 g/sq m), and the lagoon is overloaded according to design standards. Whether the overloading is the cause of the poor treatment in the polishing lagoon during the sampling period cannot be determined from the data. The probability, in the absence of additional plant operating data, that the system was temporarily upset seems more likely, based on the decreased efficiency of the mechanical plant, and the strength of the stabilization lagoon effluent.

Fecal coliform as before, showed a decrease across the treatment system with the final effluent containing no detectable fecal coliform organisms.

A second comprehensive biological investigation of the unnamed stream receiving plant effluent was not undertaken as part of the post-construction study, because the results of the pre-construction study (Appendix II) did not demonstrate the presence of any gross pollution.

Anaerobic sludge deposits were reported as occurring in the stream bed, but the investigation disclosed that the majority of the data regarding the physical and chemical nature of the stream were within the limits of tolerance of most organisms.

A general inspection of the unnamed stream was made as part of the post construction evaluation and no sludge deposits nor offensive odors were detected, although the water was colored a deep green by the algae from the lagoons.

SECTION VIII

WET WEATHER PERFORMANCE

At the conclusion of the initial extended dry weather sampling program, efforts were immediately made to begin wet weather performance evaluations. By 29-July, 1971, all sampling equipment and flow measurement devices were in place for the wet weather study. Two local Shelbyville residents were hired to change the flow recorder charts and report any unusual conditions such as equipment malfunction or vandalism. Weather conditions were monitored daily by consulting with the meteorologists at the Capital City Airport in Springfield, Illinois, located about 55 miles (88 km) from Shelbyville. When probabilities of significant rainfall in Shelbyville were predicted, a sampling crew would go to Shelbyville to operate the sampling equipment and collect grab samples.

This procedure was followed during the months of August, September, October and November with no success, due to the difficulty of predicting local rainfall occurrences with accuracy. In September additional efforts were taken to improve our procedures. The Shelbyville High School Science Department was contacted in an attempt to interest students or science club participation in the study, but no individual or organization could be found.

The services of a professional meteorological consultant and installation of the NOAA high speed weather wire were also investigated but discarded due to forecasting accuracy and time considerations. Table 11 details the initial wet weather sampling efforts.

With the onset of freezing weather in December, it was decided to remove all sampling and gauging equipment for the winter. This decision was based on the need to clean, overhaul, and calibrate the instruments. Cold weather had retarded equipment operation and increased the failure rate and need for maintenance.

This plan was followed, and the equipment was returned and installed in Shelbyville in March, 1972. It was soon discovered, however, that to obtain meaningful data it would be necessary to make a complete inspection of the combined sewage overflow bypass and diversion structures in the sewage collection system. Accumulated silt and grit in the old combined sewers had blocked some of the bypass lines and diversion structures resulting in their failure to effectively divert dry weather flows from the combined sewers.

Preceding page blank

TABLE 11

RAINFALL AND SAMPLING EFFORTS

Date	Significant Rainfall Predicted	Rainfall Recorded	Sampling Attempted	Comment
29-July	Yes	None	Yes	
10-Aug.	Yes	0.07 in.	Yes	Insufficient Rainfall
16-Aug.	Yes	None	Yes	
23-Aug.	No	0.05 in.	No	
25-Aug.	Yes	0.06 in.	Yes	Insufficient Rainfall
1-Sept.	No	0.15 in.	No	
5-Sept.	No	0.10 in.	No	
9-Sept.	Yes	None	Yes	
15-Sept.	No	0.02 in.	No	
16-Sept.	Yes	None	Yes	
18-Sept.	Yes	0.01 in.	Yes	
19-Sept.	No	0.78 in.	No	Light Drizzle
20-Sept.	No	0.23 in.	No	over 26 hr. period.
25-Sept.	No	1.40 in.	No	Light Drizzle Predicted
3-Oct.	No	0.03 in.	No	
8-Oct.	No	0.03 in.	No	
13-Oct.	No	0.05 in.	No	
14-Oct.	Yes	None	Yes	
18-Oct.	Yes	None	Yes	
21 Oct.	No	0.57 in.	No	Light Drizzle over 18 hr. period
2-Nov.	No	0.12 in.	No	
8-Nov.	No	0.02 in.	No	
18-Nov.	Yes	0.44 in.	Yes	Railfall occurred earlier than forecast. Samples were collected but discarded due to insufficient additional rainfall.
24-Nov.	No	0.05 in.	No	
26-Nov.	No	0.09 in.	No	
27-Nov.	No	0.03 in.	No	
28-Nov.	No	0.19 in.	No	Drizzle
29-Nov.	No	0.06 in.	No	

(inch x 2.54 = cm)

These structures were located and restored to service so proper wet weather operation could occur. By 5-April, 1972, the treatment system and equipment were ready for operation.

In order to assure the proper collection of data, a full-time Engineer was moved to Shelbyville to maintain the sampling and gaging equipment and initiate sample collection at the onset of wet weather. A total of 14 storms occurred during the wet weather sampling period, and the natural data and sampling activities are summarized in Table 12.

Attempts were made to sample every storm flow during the wet weather sampling period until data from at least two separate storms were obtained for each site.

The Southeast Site was the easiest to sample because almost any storm would produce enough flow to obtain adequate sampling. A total of three storms were sampled at that location, and data from two of the three could be analyzed in detail.

TABLE 12

WET WEATHER SAMPLING

Storm No.	Date	Time	Duration (HR)	Depth (IN)	Average Intensity (IN/HR)	Maximum 15 Min. Intensity (IN/HR)	SAMPLING East	SAMPLING SE	SAMPLING SW
1	4-7	7:00A to 10:00A	3	0.16	0.05	0.26	1	1	1
2	4-11	4:00P to 8:15P	4-1/4	1.25	0.29	1.20	S,2	S	S,3
3	4-12	11:45P to 3:15A	3-1/2	0.52	0.15	1.53	4	4	4
4	4-14	2:45A to 4:45A	2	0.18	0.09	0.20	1	1	1
5	4-15	6:15P to 10:15P	4	0.57	0.14	0.32	S	S	5
6	4-19	5:15P to 10:15P	5	1.23	0.24	3.40	S	S	5
7	4-21	2:00A to 5:30A	3-1/2	0.31	0.09	0.28	1	6	5
8	4-21	4:30P to 5:30P	1	0.20	0.20	0.48	1	6	5
9	4-28	2:30P to 8:00P	5-1/2	0.25	0.05	0.05	1	1	1
10	4-29	6:45A to 10:00A	3-1/4	0.23	0.07	0.45	1	6	6
11	5-4	1:00P to 5:45P	4-3/4	0.32	0.07	0.51	1	6	6
12	5-8	6:15A to 7:30A	1-1/4	0.34	0.27	0.60	1	6	S
13	5-12	11:30P to 4:30A	5	0.43	0.09	0.25	1	6	S
14	5-29	10:30P to 1:45A	3-1/4	0.62	0.18	0.95	1	6	6

NOTES:
S - Samples collected
1 - Insufficient run-off
2 - Chlorination failed, samples discarded
3 - Flow measuring equipment damaged, samples discarded
4 - Insufficient turn around time for samplers
5 - Storm Holding Lagoon (SW) full
6 - Not sampled

Wet weather sampling at the East Site was more difficult, because storms of moderate intensity and duration did not produce enough run-off to fill the storm water treatment tank. Of the 14 storms during which sampling was attempted, five produced enough run-off to cause overflow into the tank and only four produced enough run-off to fill the tank and produce an effluent. Samples were collected during three of these storms, but one set was discarded because the automatic chlorinator failed to operate midway through the storm.

The Southwest Site generally had storm flow present for any size storm, since the main combined sewer terminates at that location. Sampling was delayed at this site, however, for two reasons. First, the access road to the plant was poorly graded and heavy rainfall produced enough run-off from the surrounding agricultural land and landfill site to flood the road and, in turn, flood the influent channel and grit chamber. This flooding occurred at a location following the influent control gate, so the effect was to significantly dilute the influent raw sewage and overload the plant with suspended solids. This problem was corrected by construction of an earthen dike around the influent channel and grit chamber to prevent entry of any surface run-off.

The second problem that delayed sampling at the Southwest Site resulted from the design of the storm holding lagoon inlet structure. Combined sewage in the large interceptor flows through a bypass structure at the inlet to the lagoon. If the lagoon is full, this flow is diverted over the 28 foot (8.53 m) sharp crested rectangular weir to the unnamed creek which originates at the plant site and eventually reaches the Kaskaskia River by way of its tributary, Robinson Creek. The frequent railfall at the start of the wet weather sampling period soon filled the lagoon, and continued rainfall produced enough run-off to prevent the storm lagoon effluent pumps from lowering the lagoon water surface to the normal dry weather level. Finally, a temporary gate was constructed at the lagoon influent pipe to block all flow and allow the effluent pumps to lower the level so that further testing could be performed. Once this was accomplished, two storms were easily sampled.

Analyses performed on the wet weather samples were limited to BOD, suspended solids, and fecal coliform, since these criteria are currently of primary importance in combined sewer overflow pollution. The treatment units for this project were designed to treat these constituents, and there are no known sources of unusual pollutants such as heavy metals, organic chemicals, pesticides, oils, etc. in the City of Shelbyville.

Southeast Site - Two storms were sampled at the southeast lagoon, identified in Table 12 as numbers 2 and 6. The two storms were almost identical in total rainfall and average intensity, but storm No. 6 had a much greater maximum 15 minute intensity (3.40 in/hr - 8.63 cm/hr). The graphs of the analytical results from sampling the two storms (Figures 9 through 14) indicate that the "first flush" effect differs for BOD and suspended solids and is related to rainfall intensity, as indicated by run-off flow volume.

As would be expected, the flow to the lagoon varies with the intensity of the rainfall but lags behind the rainfall intensity peaks by only about 15 minutes. This relatively short concentration period and the direct relation between peak rainfall intensities and peak flow rates indicate that storm water run-off is reaching the lagoon very quickly, suggesting that run-off velocities are probably quite high. This observation is further supported by the fact that the differential elevation in the drainage district is about 70 feet (21 m). In comparing the suspended solids loading to the lagoon with the influent flow rates, it can be seen that the relative magnitudes are proportional, regardless of whether the peaks occurred at the beginning, middle or near the end of the storm. Thus, there is not a definite "first flush" of suspended solids occurring at the Southeast Site.

The absence of a "first flush" of suspended solids would be predicted for the southeast lagoon because of its location in a natural ravine. Overland run-off from the drainage district makes up a large portion of the wet weather flow at the lagoon influent, and the upper reaches of the natural ravine are subject to severe soil erosion. During peak run-off periods this overload flow is heavily laden with suspended solids, masking any contribution made from the combined sewer.

BOD loading to the southeast lagoon followed the first flush pattern more closely than did the suspended solids, but was still affected by the rate of flow. For storm No. 2 there was no dilution of the BOD concentration at the beginning of the run-off period, while subsequent flow peaks indicated correspondingly lower BOD concentrations. Storm No. 6 had an even better first flush with about 50% of the total BOD being delivered by only the first 10% of the total run-off. Once again, a clear and definitive first flush of BOD was partially obscured by the affect of overland run-off, although not to the same extent as were the suspended solids.

The organic loadings to the lagoon resulting from the storm flows, while high by conventional standards, were not excessive if averaged over the theoretical five days' detention

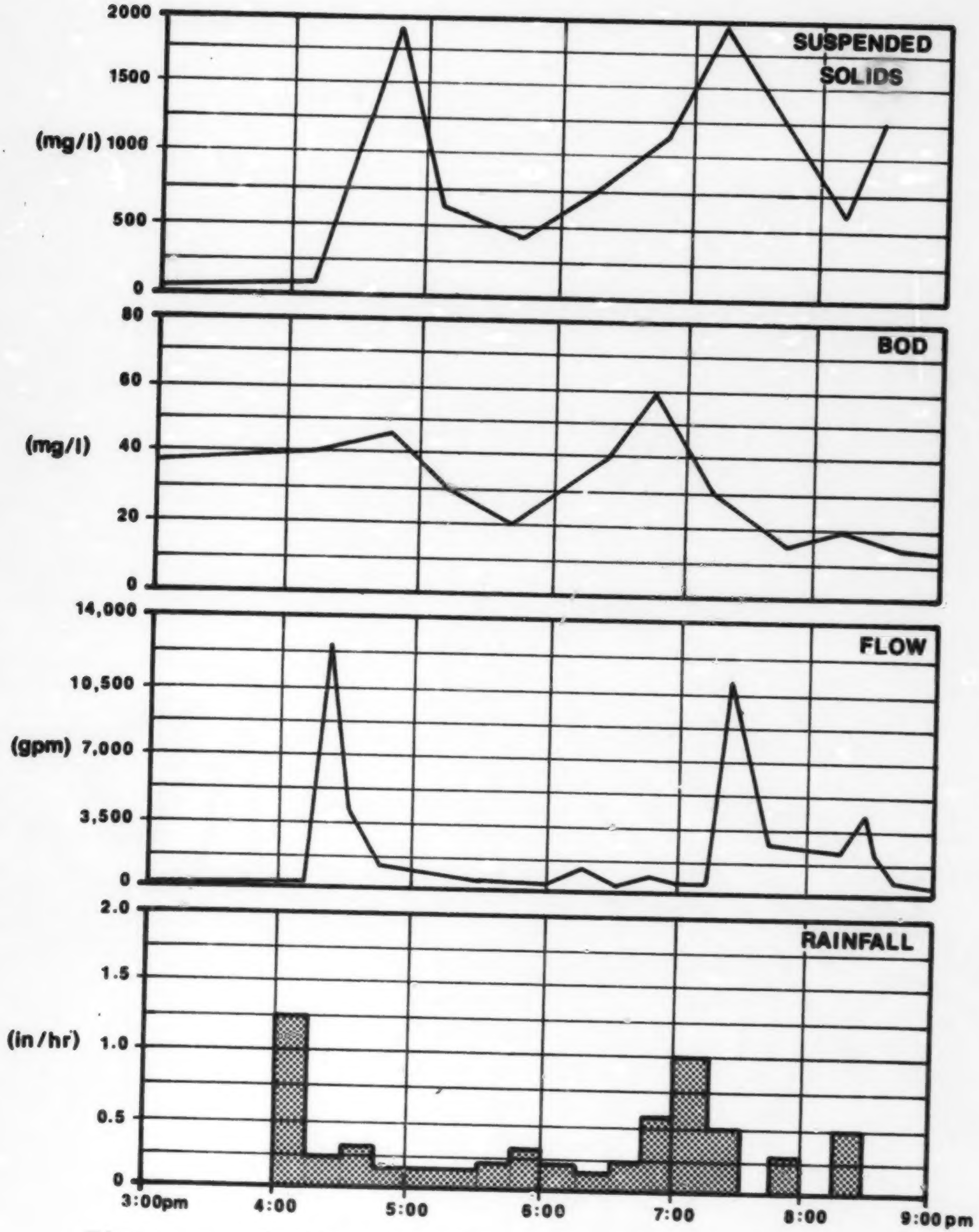


Figure 9. Southeast Lagoon Influent Natural Data
Storm No. 2, April 11, 1972
(gal x 3.785 = l; inch x 2.54 = cm)

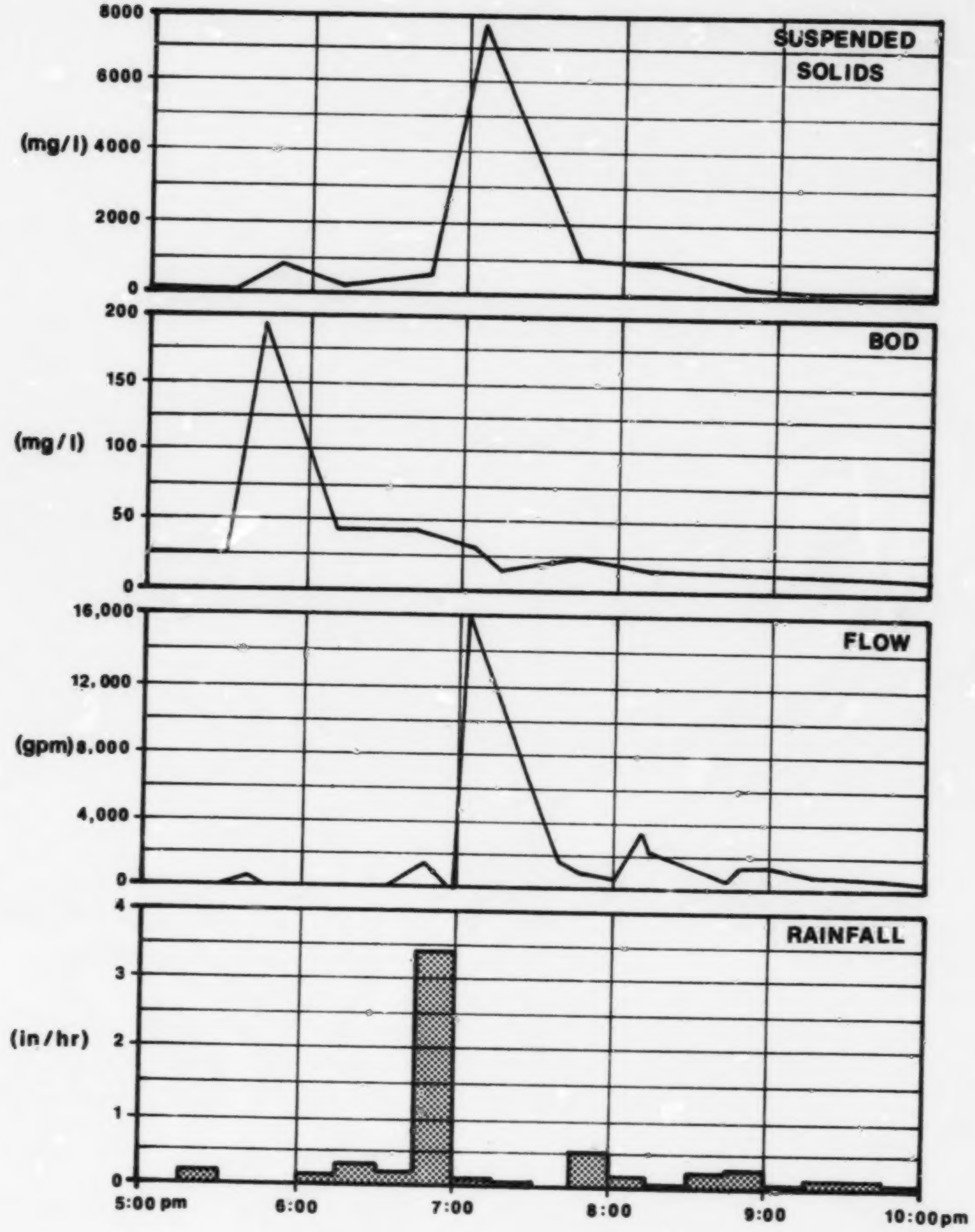


Figure 10. Southeast Lagoon Influent Natural Data Storm No. 6, April 19, 1972

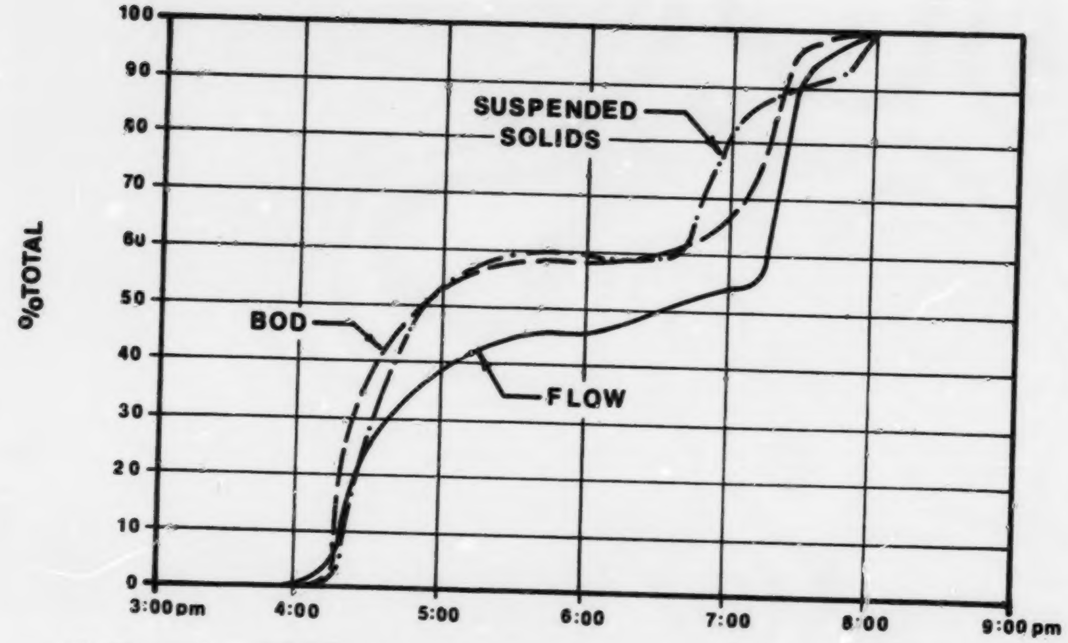


Figure 11. Southeast Lagoon Influent Cummulative Data Storm No. 2, April 11, 1972

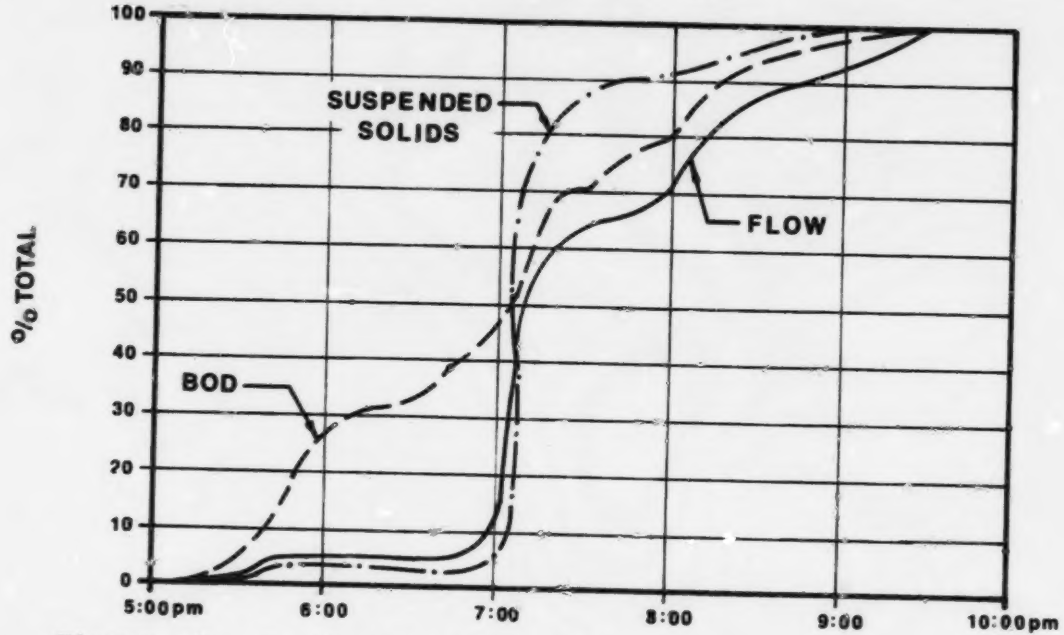


Figure 12. Southeast Lagoon Influent Cummulative Data Storm No. 6, April 19, 1972

time provided by the lagoon. For storm No. 2 a total of 130 pounds (59 kg) of BOD were applied for an average 5-day loading of 26 pounds (11.8 kg) per acre per day, or about 28 pounds (12.7 kg) per day including the dry weather flow. Storm No. 6 deposited an average of 56 pounds (25.4 kg) for a 5-day average of only about 11 pounds (5 kg) or 13 pounds (5.9 kg) per acre per day including the dry weather flows.

Grab samples were taken of the lagoon effluent flow from the start of storm No. 2 until 5 days following storm No. 6 (Figure 13). The samples were collected at approximately 12 hour intervals and more frequently during the two storms.

The results of the BOD and suspended solids analyses indicate that there is some initial carry-over during the period immediately following the storm flow but that after about 12 hours the effluent BOD and suspended solids levels drop considerably. Following the storm flows, BOD values dropped to below 6 mg/l and suspended solids varied between 50 and 75 mg/l. There was no apparent ill effect due to the high BOD loads from the storms other than the initial carry over.

Suspended solids in the lagoon effluent decreased more slowly after periods of peak influent flow, presumedly because fine clay particles washed into the lagoon from the upper reaches of the natural ravine required a long detention time to settle out.

The effect of several other storms occurring during the period of lagoon evaluation could not be studied in detail because of limitations of the sampling equipment. The effect of these other storms can be seen, however, in the lagoon effluent flow rates, Figure 13. Of special interest is the cumulative mass plots of BOD, suspended solids and flow (Figure 14). If the brief periods of carry-over resulting from the storms not samples are neglected, the nearly linear cumulative BOD plot yields an average effluent BOD of slightly more than 5 mg/l. Suspended solids do not average out nearly as well (20 mg/l) but again the cause is believed to be the clay particles washed into the lagoon, plus some contribution from algal growth.

East Site - Samples from the first storm producing sufficient run-off at the east site treatment unit were collected during the Storm identified as No. 2 in Table 12, but a malfunction in the chlorination system occurred and the samples were discarded. After repairs were made, two storms were easily sampled on the 15th and 19th of April, and identified as No. 5 and No. 6 in Table 12. Storm No. 5 (Figure 15) differed from No. 6 in that it gradually built from light to moderate

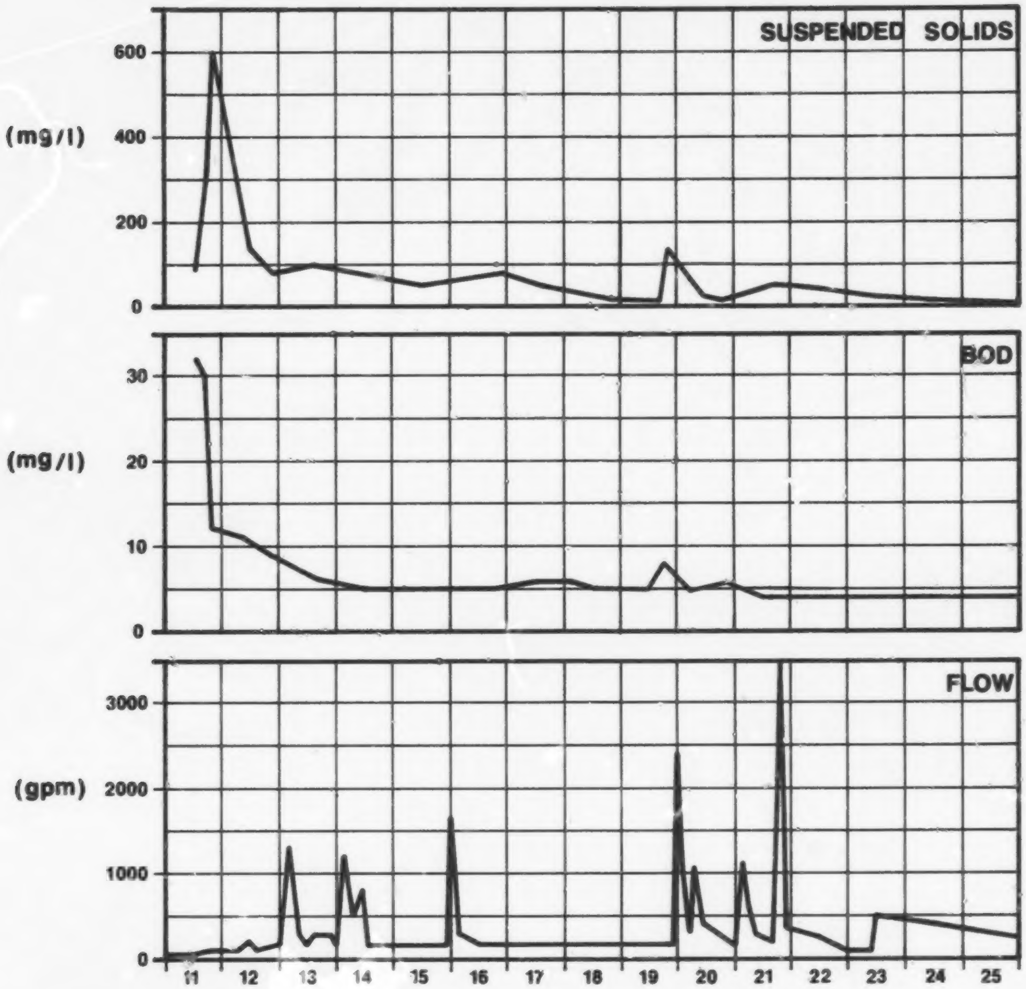


Figure 13. Southeast Lagoon Effluent Natural Data
April 11 to April 25, 1972
(gal x 3.785 = l)

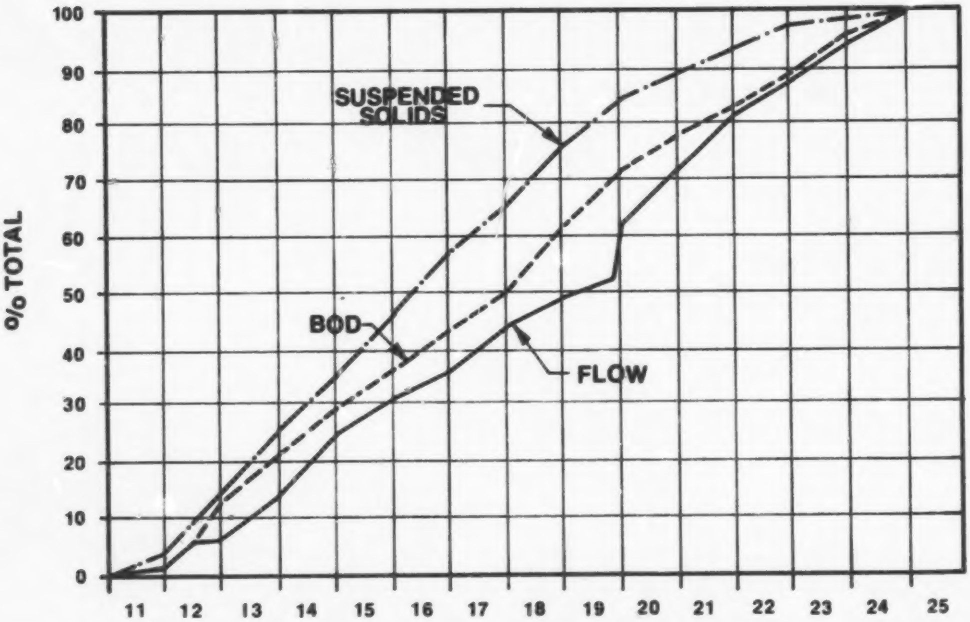


Figure 14. Southeast Lagoon Effluent Cummulative Data
April 11 to April 25, 1972

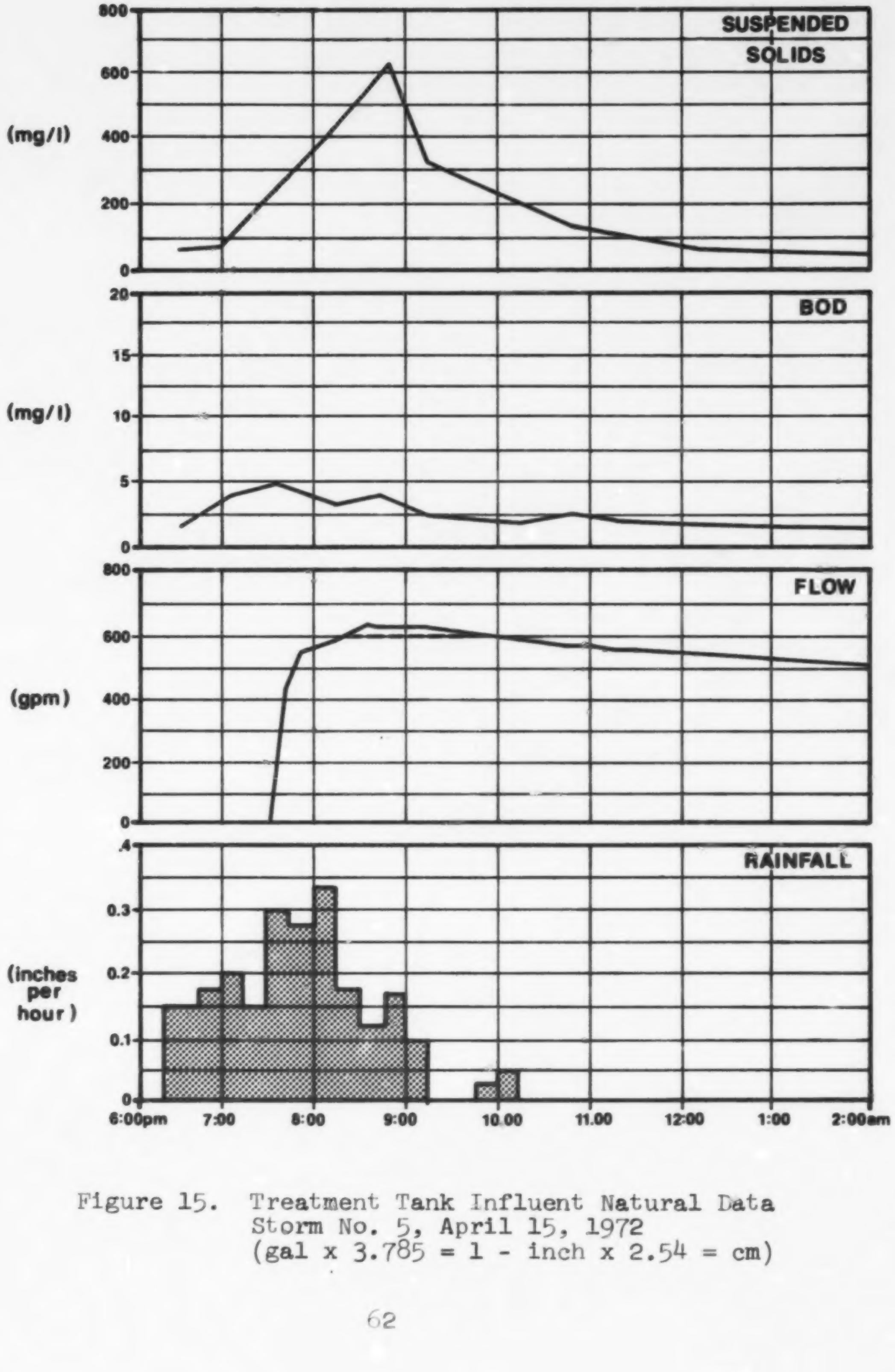


Figure 15. Treatment Tank Influent Natural Data
Storm No. 5, April 15, 1972
(gal x 3.785 = l - inch x 2.54 = cm)

intensity, and then declined to a light intensity before stopping. Rainfall was continuous and there were no high intensity peaks as seen in Storm No. 6 (Figure 16).

The design of the East Site treatment facilities is such that unless the early storm run-off is very intense it is possible that the first flush will be pumped by the sewage lift station into the sanitary sewer system. This, of course, is due to the fact that until the pump station capacity is exceeded, no flow will enter the treatment tank. It is also readily observed that in order to operate as a flow through treatment unit, run-off must be of sufficient duration to not only exceed pump capacity, but also fill the tank, otherwise excess run-off is merely retained in the tank. The available storage capacity of the system is not great, the treatment tank will hold about 52,900 gallons (200,000 l) and the influent sewers and manholes add another 12,500 gallons (47,300 l) for a total of 65,400 gallons (247,300 l). For light rainfalls, where run-off is not great, the storage capacity available in the system becomes a significant part of the treatment process, tending to equalize a peak flow.

As indicated in the rainfall data in Table 12, of the 14 storms, only four produced enough run-off to even test the treatment system. The rest were either handled by the pump station or only partially filled the treatment tank. Since the run-off is not only a function of the rainfall intensity and duration, but also the condition of the ground surface (i.e. frozen, saturated, etc.) there is no particular intensity or duration of rainfall that will provide a known amount of run-off at all times.

In attempting to evaluate the facilities at the East Site in terms of treatment afforded the first flush, an artificial condition was created during Storm No. 5 by shutting off the sewage lift pumps. This had the effect of increasing the BOD and suspended solids loading to the treatment tanks thus simulating a more intense storm in which the run-off quantities at the start of the run-off period would be much greater than those expected for Storm No. 5. This calculated modification of the treatment process was made to provide a better test of the system in the event subsequent rainfall was not of sufficient intensity. Fortunately the second storm sampled was of sufficient intensity, however, and the system could be evaluated under design conditions.

The influent sampling was performed at the first manhole upsewer from the treatment tank, thus indicating the suspended solids and BOD concentrations of the flow handled by the pump station prior to filling the tank. This was done to demonstrate that the first flush at this site was handled, in part,

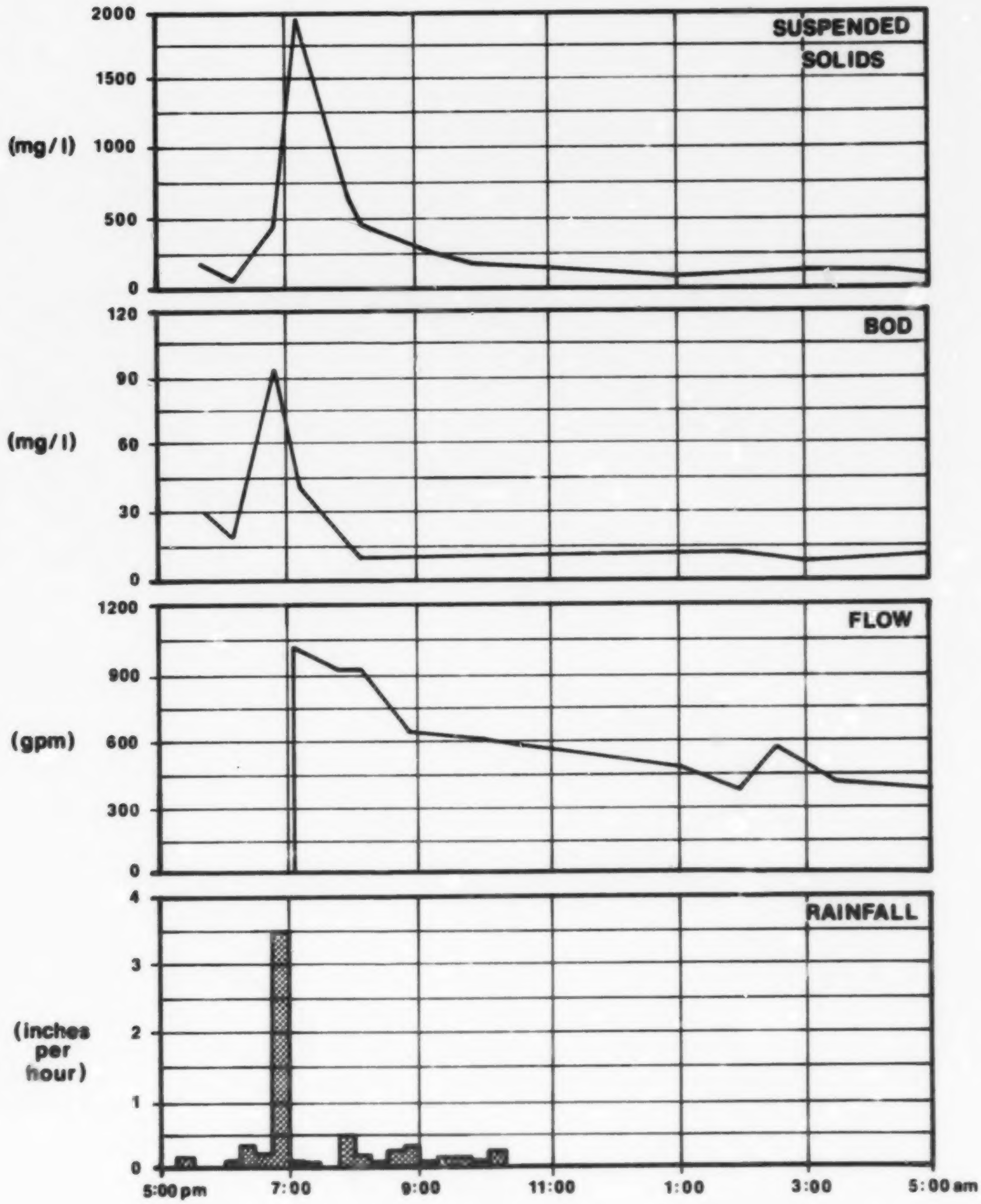


Figure 16. Treatment Tank Influent Natural Data
Storm No. 6, April 19, 1972
(gal x 3.785 = l; inch x 2.54 = cm)

by the pump station prior to flow entering the treatment tank.

The graphic representation of the data from storm No. 5 is shown in Figures 15, 17, and 19. Both the BOD and suspended solids loadings show a definite first flush at the start of the run-off period, with the BOD peak preceeding that of the suspended solids. The portion of the first flush entering the treatment tank, however, is attenuated because the initial flows were handled by the pump station. After the tank fills, there is a slight peak in the initial BOD discharged (probably due to any initial floating matter or contamination in the effluent trough) followed by a gradual decline in effluent BOD and suspended solids.

The process efficiency, in terms of pounds removed during Storm No. 5, was 61% for BOD and 53% for suspended solids.

The second storm sampled at the East Site was Storm No. 6. With this storm, the treatment unit was entirely under automatic control and the lift station pumps operated as required.

The sampling data is shown graphically in Figures 16, 18, and 20. As can be seen, there was a very sharp-peaked first flush with the BOD peak again occurring before the suspended solids peak. The peak BOD concentration occurred just prior to flow into the treatment tank, while the peak suspended solids concentration occurred just after. After the tank fills there again is an initial small BOD peak in the effluent with an average decrease in BOD with time. Suspended solids concentration in the effluent flow decreased at a gradual rate to a fairly constant residual of 50-75 mg/l.

The process efficiency of the system in terms of pounds removed was similar to the previous storm. BOD removal averaged 65.5% and suspended solids removal averaged 53%.

Fecal coliform results, although not shown graphically, varied randomly in the influent from 900 to 430,000 MPN/100 ml, with the higher values occurring at the start of the run-off period. No attempt was made to adjust the chlorination rate during the storm periods. The equipment was set to provide a dose of 18 mg/l for the first storm and 25 mg/l for the second storm at a flow of 1.6 MGD (6.05 MLD), and all effluent samples tested had no detectable fecal coliform.

Southwest Site - The first storm samples at the Southwest Site occurred 8-May - 1972 and is identified on Table 12 as No. 12. The storm began at moderate intensity, and decreased

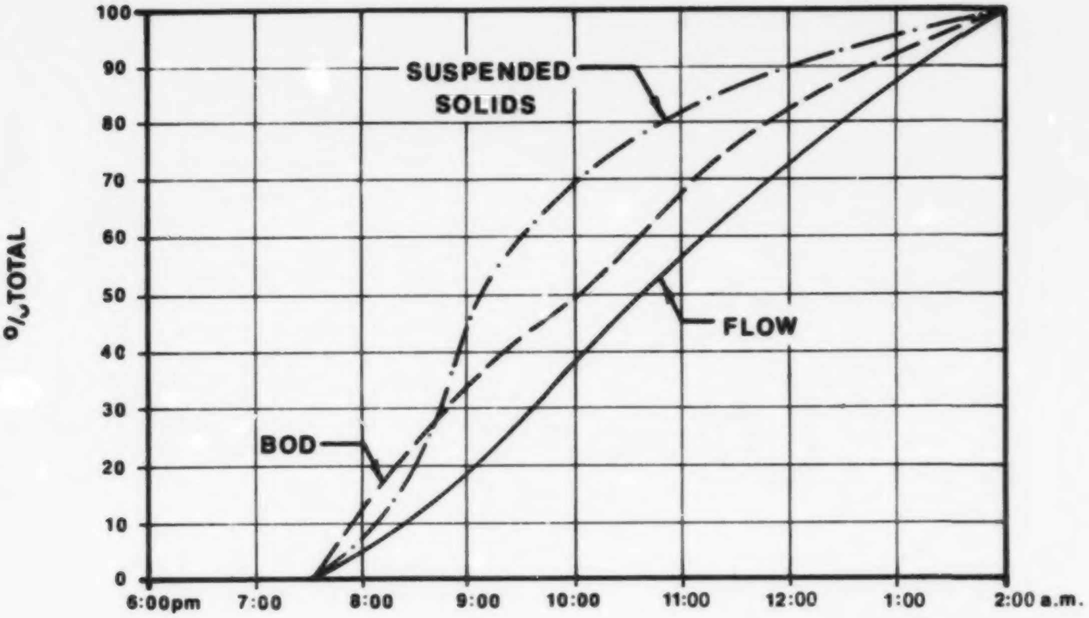


Figure 17. Treatment Tank Influent Cummulative Data
Storm No. 5, April 15, 1972

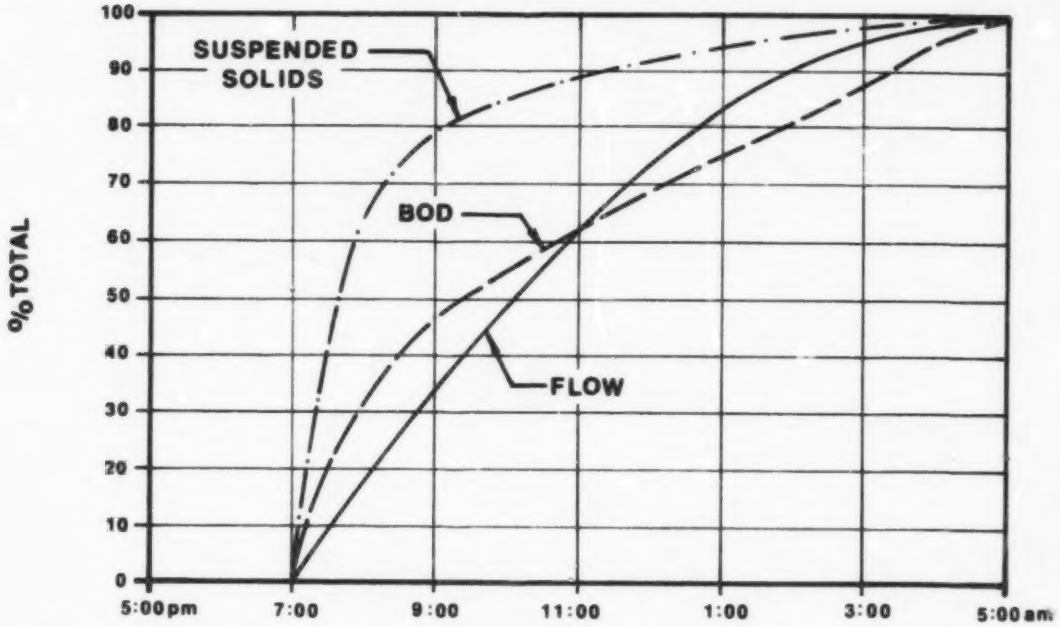


Figure 18. Treatment Tank Influent Cummulative Data
Storm No. 6, April 19, 1972

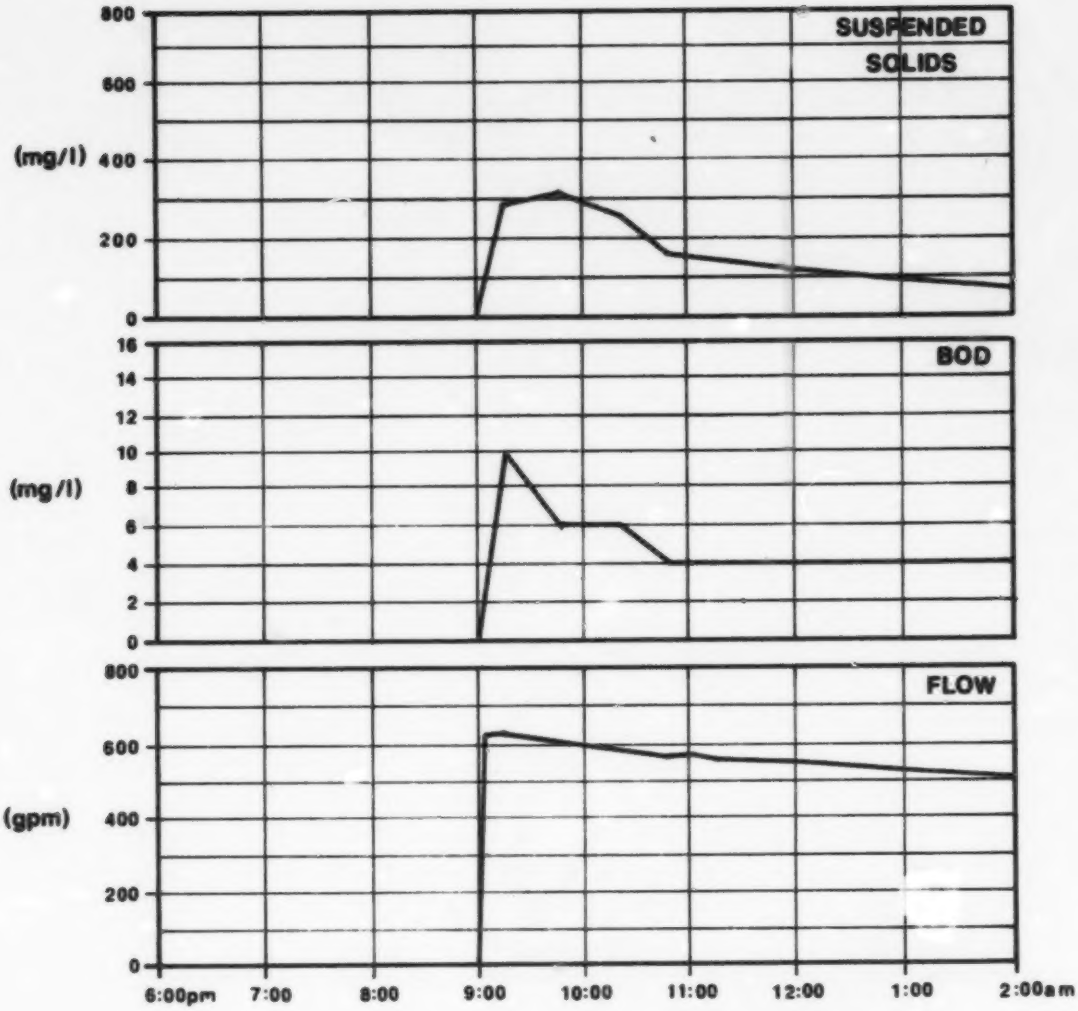


Figure 19. Treatment Tank Effluent Natural Data Storm No. 5, April 15, 1972. (gal x 3.785 = 1)

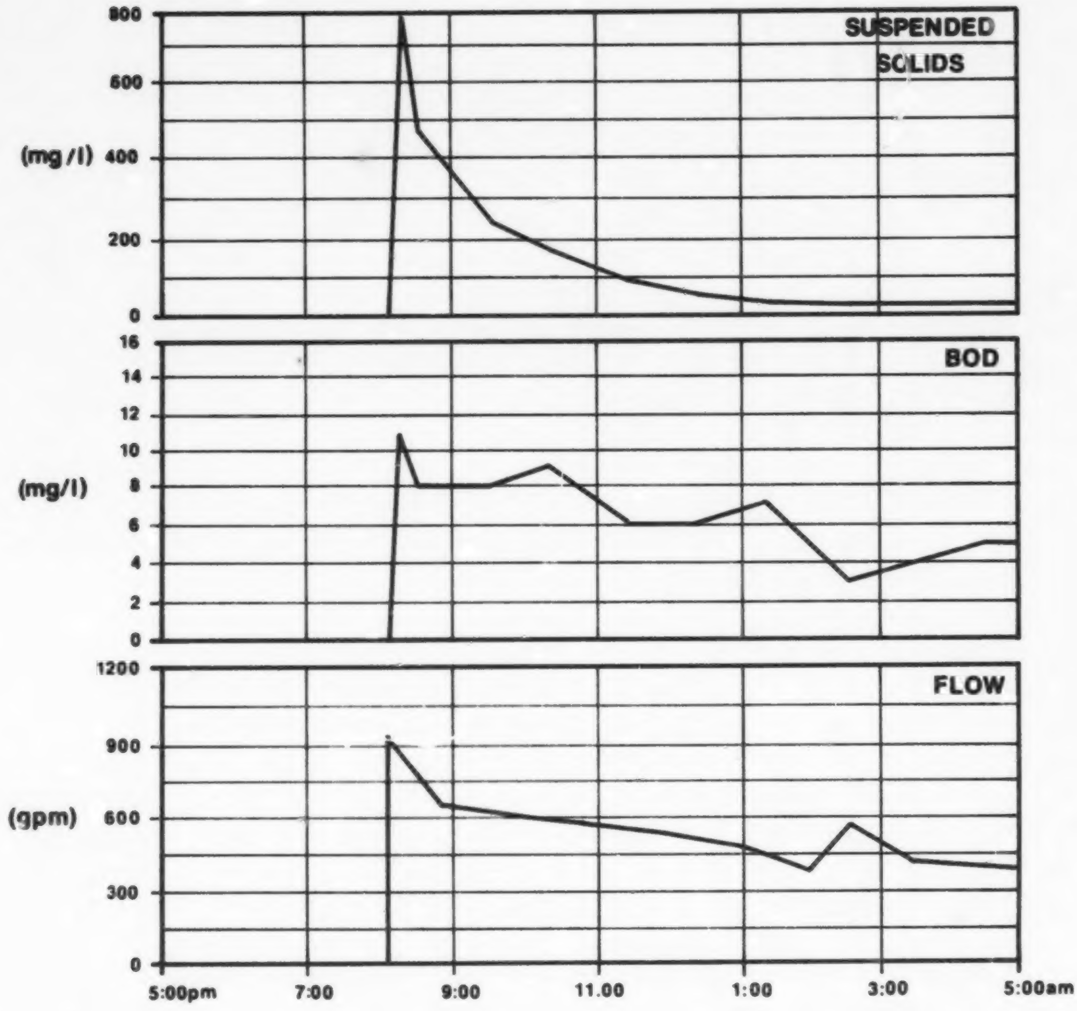


Figure 20. Treatment Tank Effluent Natural Data
Storm No. 6, April 19, 1972
(gal x 3.785 = l)

to a drizzle after 45 minutes. Total depth of the rainfall was 0.34 inches (.854 cm).

As shown in Figures 21 and 22, the rainfall at the start of the storm produced a sharp hydraulic peak at the plant as well as at the storm lagoon influent. BOD and suspended solids also peaked at the start of the storm at both plant and lagoon, but the BOD concentration at the plant influent was nearly twice as concentrated as that entering the storm lagoon. After the initial peak, both BOD and suspended solids in the plant influent decreased, but did not become as dilute as the flows entering the holding lagoon. Plant effluent analyses did not indicate any degradation in plant effluent resulting from the storm, despite the abnormally high suspended solids loading. Flow into the plant was limited by the influent sluice gate valve to prevent flooding of the comminutor.

The excess flow was bypassed to the storm lagoon at the plant influent bypass structure, and the effect of this bypassing can be seen by the increase in BOD concentration of the storm lagoon influent after the storm had subsided. Increased maintenance and attention to the grit chamber and comminutor are required to recover some of the lost hydraulic capacity of the comminutor, and permit higher flows to enter the plant during wet weather.

Total loading to the storm lagoon is shown by the cummulative curves in Figure 23. About 900 pounds (410 kg) of BOD and 4600 pounds (2100 kg) of suspended solids were applied to the lagoon, and the average depth increased from 45 inches (114 cm) to 62 inches (151 cm). As this depth increase was not sufficient to cause the storm lagoon effluent pumps to automatically start, all of the overflow applied to the lagoon was retained for a period of three days. At the end of retention period, the storm lagoon effluent pumps were operated for a period of 14 hours and pumped about 880,000 gallons (3.33 ML) from the storm lagoon into the stabilization lagoon, lowering the storm lagoon level to 54 inches (137 cm).

The second storm, No. 13 in Table 12, occurred on 13-May, 1972. Unlike the previously sampled storm, rainfall began as a light drizzle for the first two hours with no bypassed flow and only slight increases in plant influent flow. After the period of drizzle, the intensity increased moderately, delayed to drizzle and stopped briefly, and concluded as a brief shower of moderate intensity.

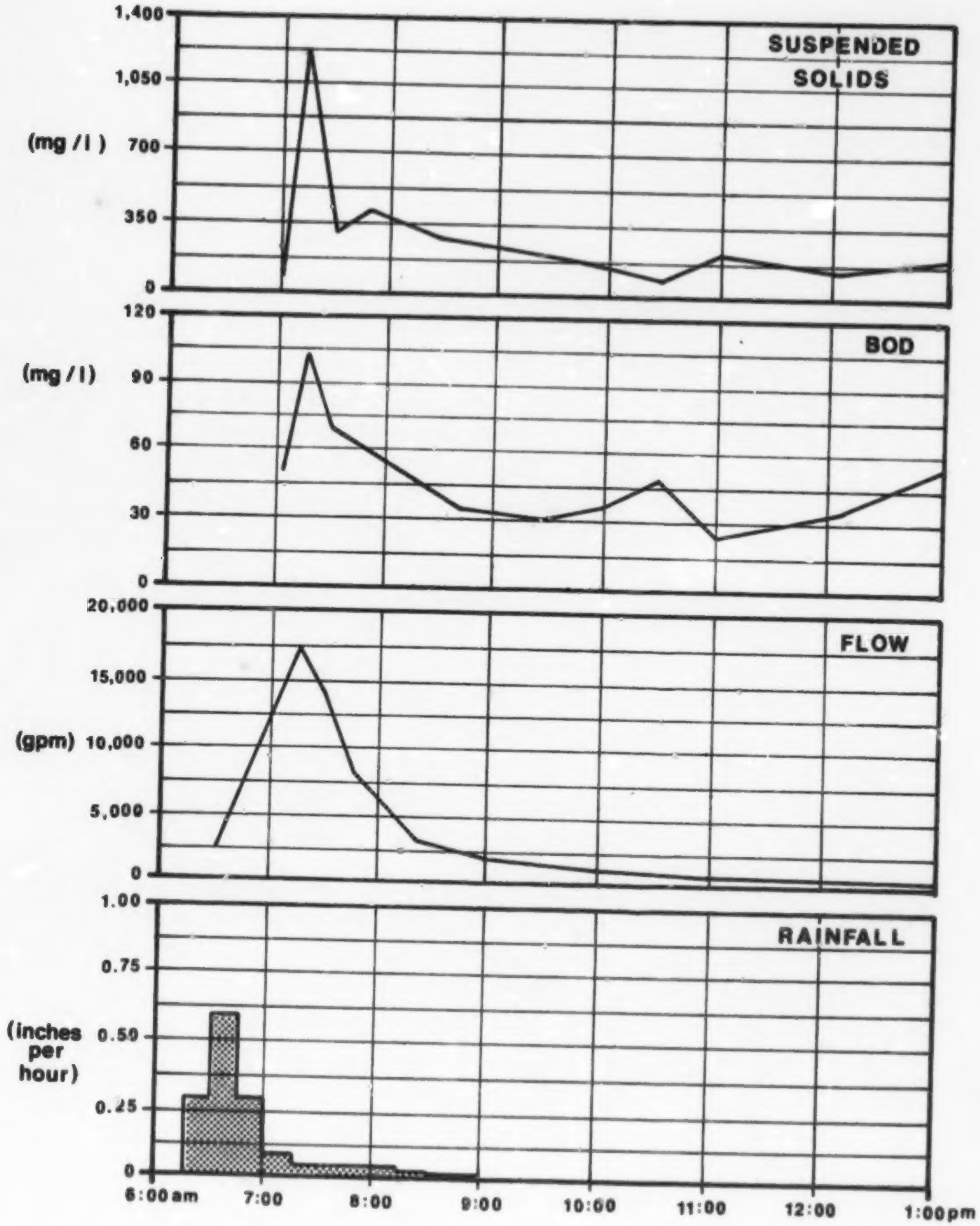


Figure 21. Storm Holding Lagoon Influent Natural Data Storm No. 12, May 8, 1972 (gal x 3.785 = l; inch x 2.54 = cm)

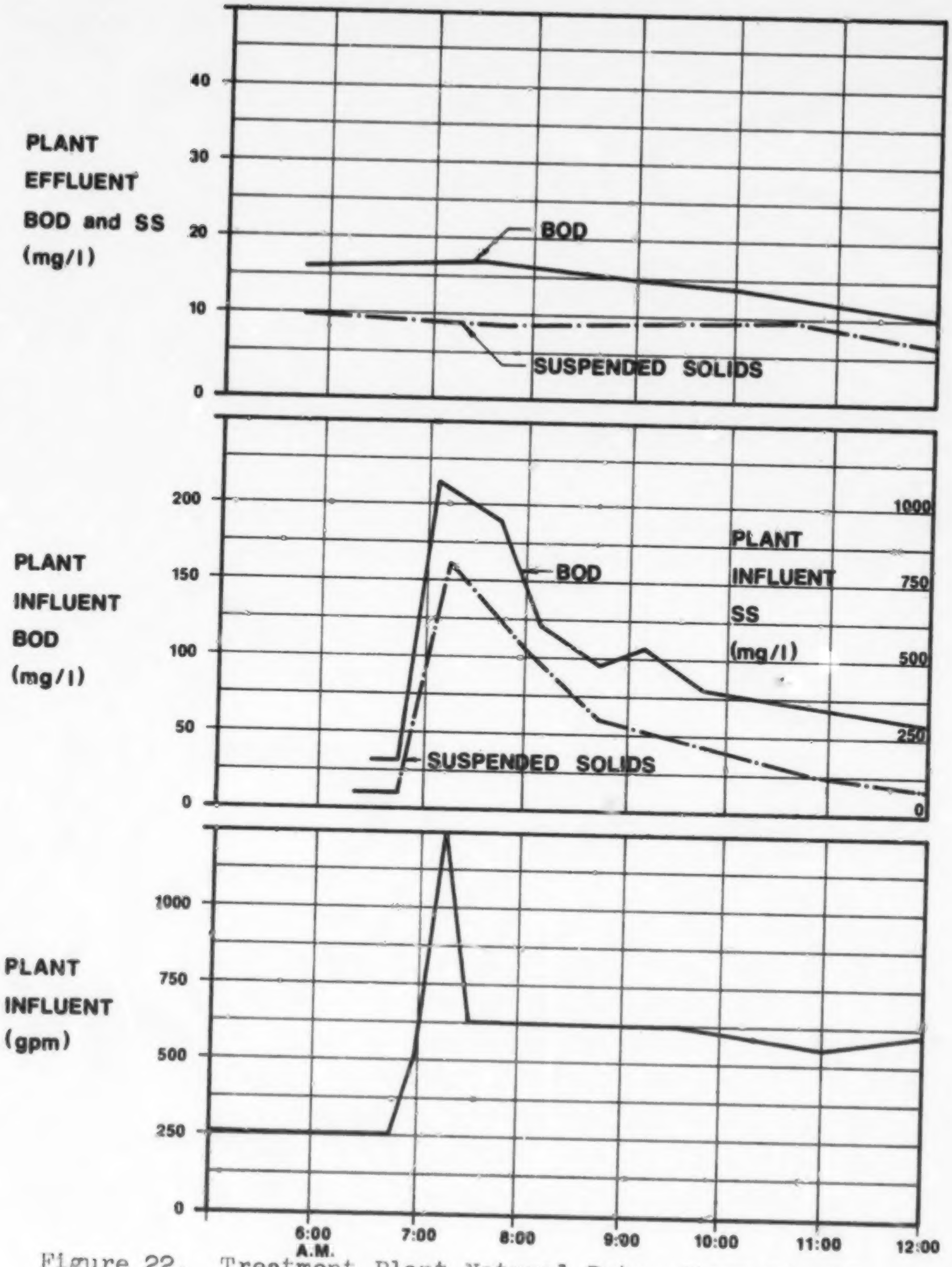


Figure 22. Treatment Plant Natural Data, Storm No. 12 May 8, 1972 (gal x 3.785 = l)

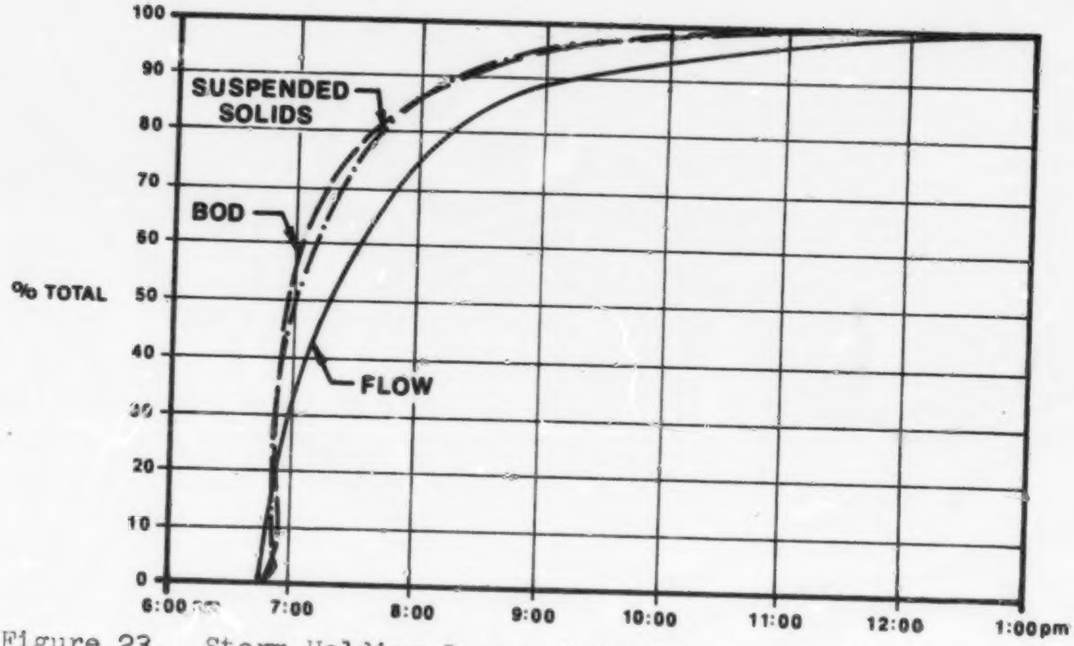


Figure 23. Storm Holding Lagoon Influent Cummulative Data
Storm No. 12, May 8, 1972

Combined sewer overflow into the storm holding lagoon began after the first period of moderate intensity rainfall, and continued throughout the remainder of the storm, with a second peak coincident with the second peak of rainfall intensity (Figure 24). Unlike the earlier storm, plant influent flow did not reach as high a rate during the peak overflow period. A first flush effect was observed in the plant influent suspended solids and BOD levels during the overflow from the first period of peak rainfall and was absent during the run-off from the second period of peak rainfall. Again the concentration of BOD and suspended solids going to the plant was considerably greater than that entering the storm lagoon during peak flow. Plant effluent during the storm had a negligable improvement in BOD and moderate increase in suspended solids, but the values obtained were within a range of expected values for normally functioning trickling filters.

Flow entering the storm holding lagoon occurred in two peaks, corresponding to the peaks in rainfall intensity. Suspended solids and BOD rates (Figure 24) tended to follow the flow rates in magnitude but were significantly attenuated during the second peak run-off flow indicating that a true first flush had occurred in the system.

Total loading to the storm lagoon was less from the second storm. Only 165 pounds (75 kg) of BOD and 620 pounds (280 kg) of suspended solids were deposited in the storm lagoon, and the level of the lagoon increased by only six inches (15 cm), to a total depth of 60 inches (150 cm).

Again, the total overflow added to the holding lagoon was not sufficient to start the lagoon effluent pumps automatically so the entire contents of the lagoon was retained for a period of four days before the effluent pumps were manually started, and allowed to pump to automatic shut-off. Pumping continued for 34 hours with a total pumpage of 1,034,000 gallons (3.92 ML).

Four samples of the storm holding lagoon contents were taken twice daily about eight feet from the water line and about 100 feet (30 m) from each corner and composited to get a daily sample of average lagoon contents. While the effluent pumps were operating, the samples were collected at the pump station. A plot of these samples is shown in Figure 27.

As can be seen from the Figure, during periods of storm flow, the BOD and suspended solids fluctuate as the material enters the lagoon and mixes with the initial contents. During the

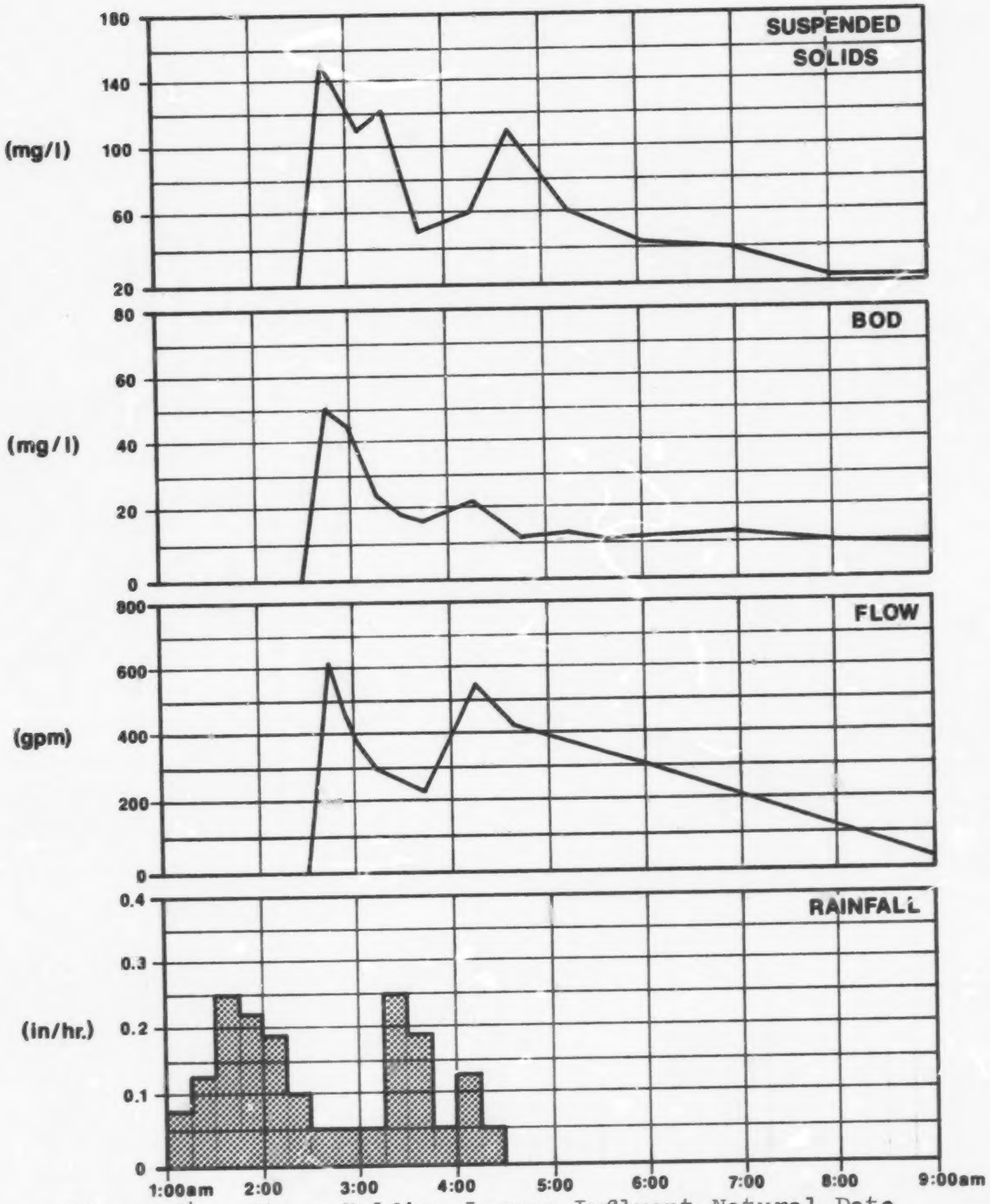


Figure 24. Storm Holding Lagoon Influent Natural Data
Storm No. 13, May 12, 1972
(gal x 3.785 = l; inch x 2.54 = cm)

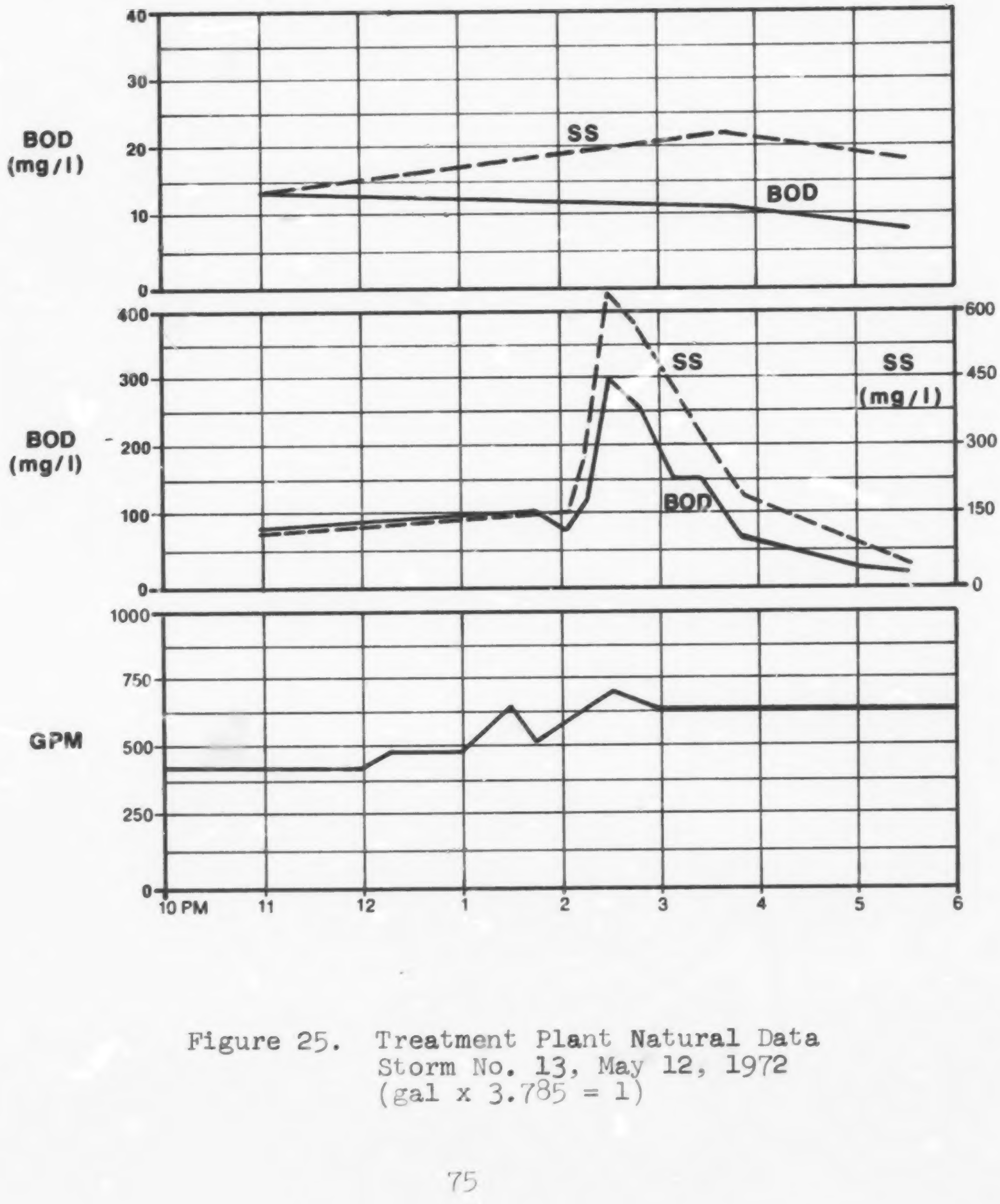


Figure 25. Treatment Plant Natural Data
Storm No. 13, May 12, 1972
(gal x 3.785 = l)

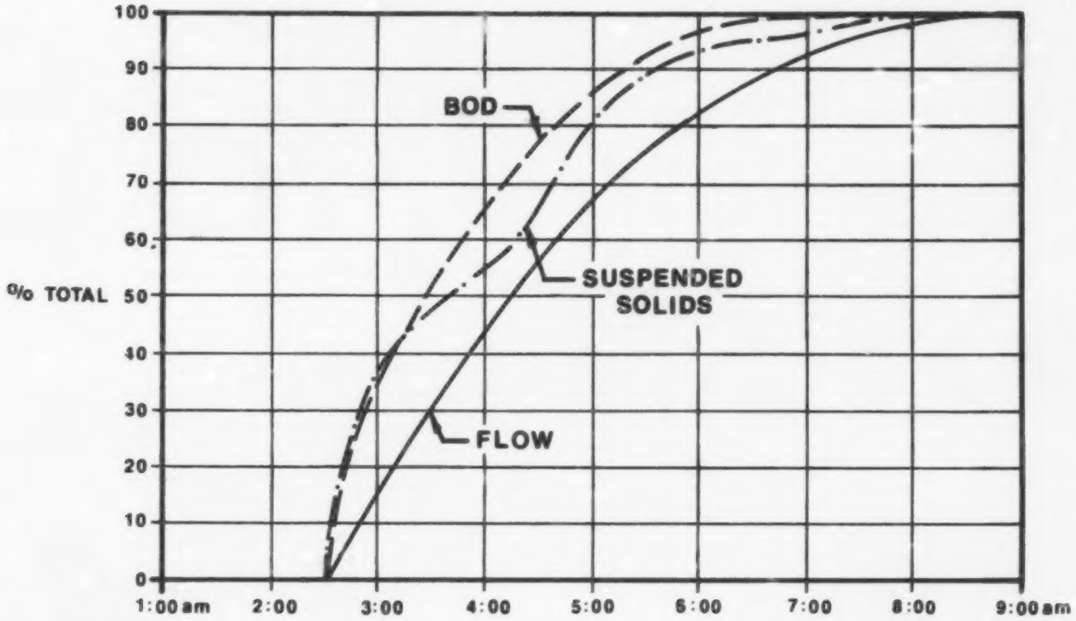


Figure 26. Storm Holding Lagoon Influent Cummulative Data
Storm No. 13, May 12, 1972

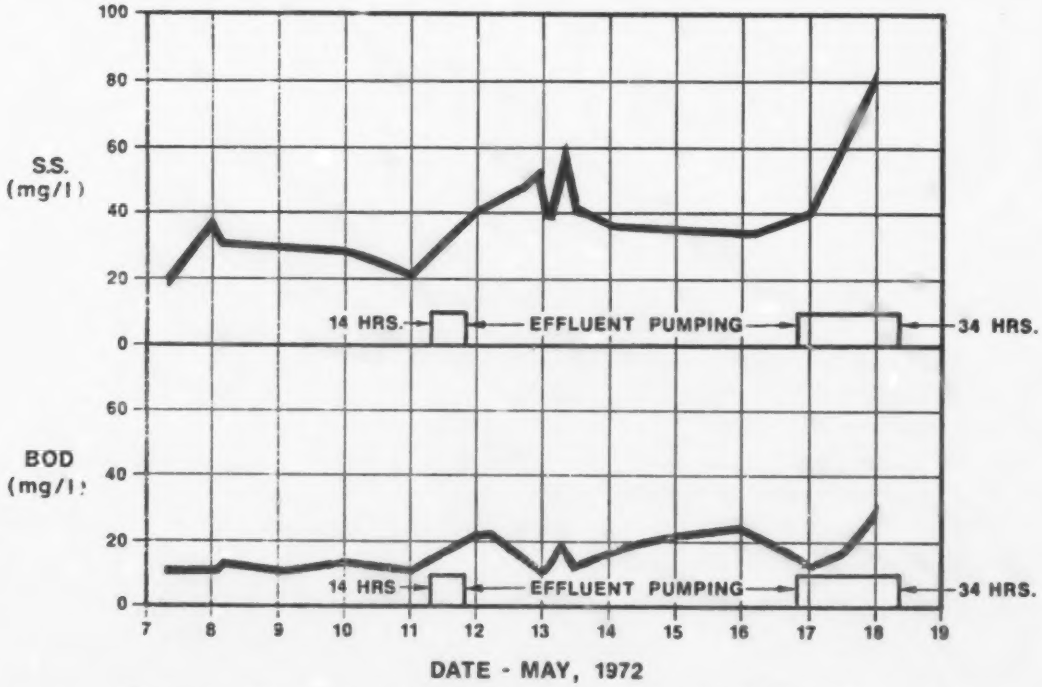


Figure 27. Storm Holding Lagoon Contents

retention period after the first storm, very little significant change occurred to the average BOD and suspended solids. Again during the retention period following the second storm there were no great changes in parameters.

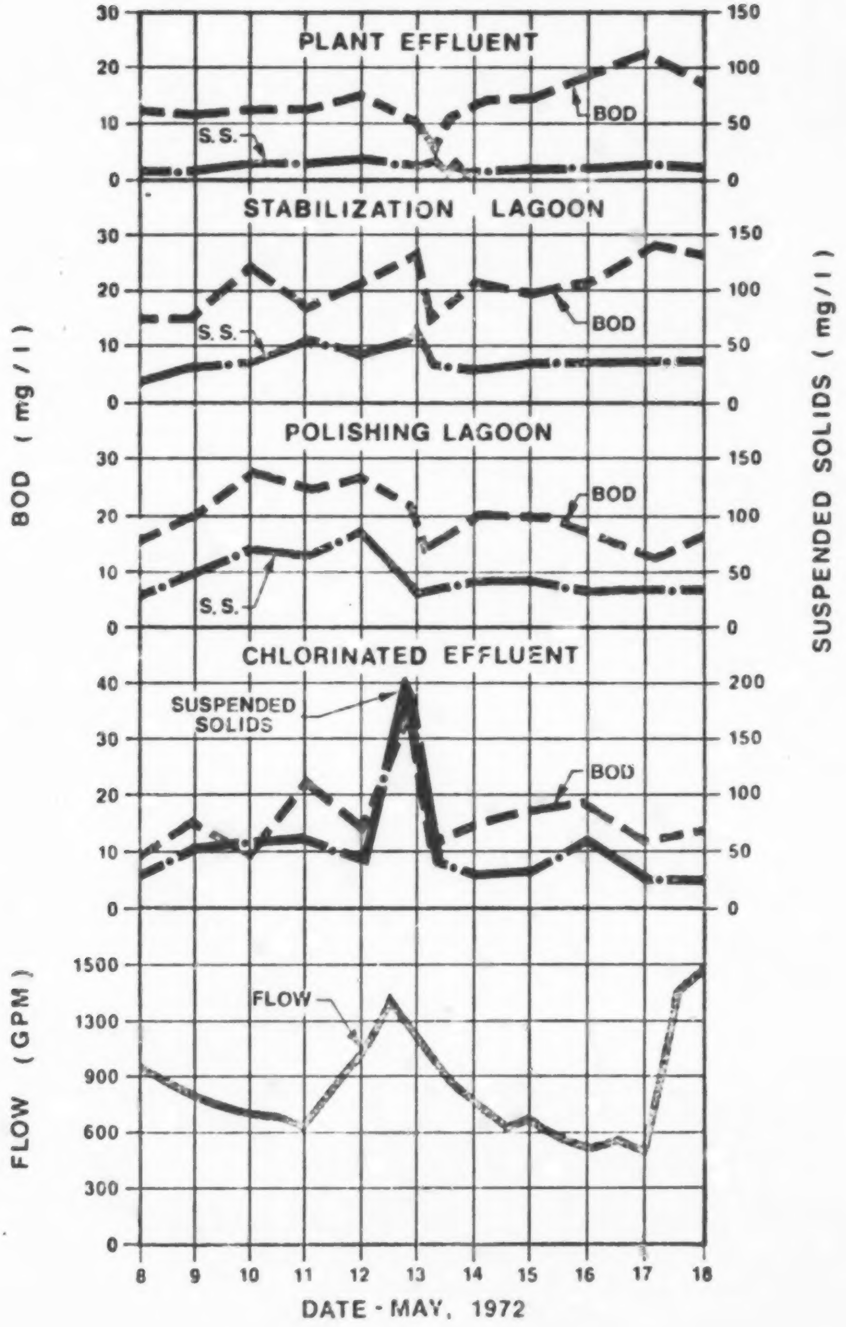
Samples taken at the pump station, while the pumps were operating, tend to have higher BOD and suspended solids values than the average lagoon contents. This is believed due, in part, to bottom scour of the sediment in the lagoon as it flows into the pump station wet well. While the values do tend to make the contents of the lagoon appear more concentrated in BOD and suspended solids, at the same time they provide a more accurate analysis of the loading applied to the stabilization lagoon.

BOD, suspended solids, and flow data for the tertiary lagoon system are shown in Figure 28, for the period of wet weather sampling at the Southwest Site. All influent to the tertiary lagoon system is either pumped from the trickling filter plant effluent or from the storm holding lagoon; natural drainage from the land around the Southwest Site cannot enter the lagoons.

Variation of the BOD and suspended solids in the effluents of the lagoons and final chlorinated effluent do not show evidence of any recognizable trends, other than that due to algal growth fluctuations. The peaks of suspended solids between the 12th and 13th of May occur during periods of higher than average flow, due to pumping from the storm holding lagoon and increased plant flow from the run-off of Storm No. 13, yet during the second period of pumping from the storm lagoon, flows were equally as great but there were no peaks in suspended solids.

The BOD loadings to the tertiary treatment system are summarized in Table 13. The tabulated data indicates that the organic loading to the stabilization lagoon was not excessive and only once exceeded the design value of 26 pounds 11.8 kg) of BOD per acre per day. Effluent BOD values from the stabilization lagoon were in every case higher than influent BOD's suggesting that algae growth was a significant factor in the lagoon performance.

Loading values for BOD applied to the polishing lagoon are much higher than the design values, however it must be considered that much of the measured BOD is from the algae and does not represent a true load.



TERTIARY POND PERFORMANCE
FIGURE 28

Final effluent values, again, reflect the presence of prolific algal growth, especially with respect to suspended solids. In comparison to data from dry weather periods, especially the more recent 24-hour dry weather sampling, the lagoon system is producing an average effluent regardless of whether the influent is derived from the treatment plant, or the treatment plant and holding lagoon together.

TABLE 13
AVERAGE BOD LOADING - TERTIARY TREATMENT LAGOON SYSTEM

Date	Stabilization Lagoon Influent	Polishing Lagoon Influent	Polishing Lagoon Effluent	Final Effluent (Chlorinated)	Stabilization Lagoon Loading	Polishing Lagoon Loading
8-Mar.	100 lbs/day	120 lbs/day	150 lbs/day	100 lbs/day	9.2 ppA/day	60 ppA/day
9-Mar.	70 lbs/day	180 lbs/day	200 lbs/day	130 lbs/day	6.4 ppA/day	90 ppA/day
10-Mar.	80 lbs/day	200 lbs/day	240 lbs/day	100 lbs/day	7.3 ppA/day	100 ppA/day
11-Mar.	140 lbs/day	150 lbs/day	200 lbs/day	180 lbs/day	12.8 ppA/day	75 ppA/day
12-Mar.	250 lbs/day	350 lbs/day	320 lbs/day	290 lbs/day	22.9 ppA/day	175 ppA/day
13-Mar.	80 lbs/day	250 lbs/day	280 lbs/day	260 lbs/day	7.3 ppA/day	125 ppA/day
14-Mar.	80 lbs/day	200 lbs/day	190 lbs/day	190 lbs/day	7.3 ppA/day	100 ppA/day
15-Mar.	60 lbs/day	150 lbs/day	150 lbs/day	150 lbs/day	5.5 ppA/day	75 ppA/day
16-Mar.	90 lbs/day	170 lbs/day	110 lbs/day	120 lbs/day	8.3 ppA/day	85 ppA/day
17-Mar.	200 lbs/day	220 lbs/day	60 lbs/day	150 lbs/day	18.4 ppA/day	110 ppA/day
18-Mar.	300 lbs/day	360 lbs/day	130 lbs/day	190 lbs/day	27.5 ppA/day	100 ppA/day

SECTION IX

DISCUSSION

In evaluating the performance of the combined sewage treatment units constructed for this project, two limitations are immediately apparent. First, only two storms were sampled at each treatment unit. The data collected, while believed to be both reliable and accurate, is not statistically sufficient to provide an accurate appraisal of the average unit performance. Variations in run-off rates, quantities, and strengths are significant considerations that can only be evaluated after study of additional storms at each treatment unit.

The second major limitation of this study lies in the fact that all storms occurred in a one-month period, 11-April to 12-May, 1972. It would be expected that for a given storm there would be distinct seasonal variations in run-off characteristics and lagoon performance. These variations may be anticipated in a general sense, but additional data would be required for a detailed analysis.

These two limitations are not considered to be restrictive when the data from this project are considered along with the other available literature regarding lagoon performance and sedimentation theory. The various treatment processes were evaluated along comparative lines and caution should be exercised in interpreting the results too broadly. If it is desired to make use of the findings of this study, the limitations mentioned must be considered. The statistical reliability of a small sample should be recognized, especially when variations in expected field conditions are known to exist.

With respect to the forementioned limitations of this study, further discussion will be limited to design considerations and a general performance evaluation based only on the data at hand. It would be expected that results of this study will be useful in evaluating further design efforts in conjunction with data from other related projects.

Sewer Separation - The partial separation of the combined sewers provided considerable improvement as seen by the difference in sewage strengths of flows entering the plant and flows entering the storm holding lagoon at the Southwest Site. Formerly, of course, wet weather plant flow and bypassed storm flow were derived from the same combined sewer and were identical in characteristics. After improvements, flow to the plant had a BOD from two to seven times greater

Preceding page blank

than flow to the storm holding lagoon during peak run-off. This had the dual effect of providing secondary treatment to more organic material as well as maintaining an adequate organic load to the plant during wet weather.

Suspended solids in the peak run-off flows to the plant and storm holding lagoon varied according to the velocity of flow in the combined sewer. For Storm No. 12, with high run-off flow rates, and in turn scour velocities, more suspended solids were transported, and peak concentrations entering the plant were somewhat less than that entering the lagoon. Storm No. 13 had some lower run-off flow rates, and peak suspended solids entering the plant were about four times greater than that entering the lagoon. Of further interest is the fact that Storm No. 13 had two separate periods of peak run-off flow which produced two corresponding suspended solids peaks in the combined sewage flows entering the lagoon, but only a first flush peak in the flow entering the plant.

The behavior of suspended solids can be explained by the fact that the hydraulic capacity of the separated sanitary system entering the plant is not nearly as great as that of the combined sewers, thus a given storm should produce higher velocities in the separated system which in turn will transport more solids. The separated system is subject to a higher velocity first flush which in the storms sampled had a thorough cleaning effect. The combined sewers, however, are heavily laden with sediment and subject to additional solids influx with each period of surface run-off. Thus a flush of solids may be expected with each peak in run-off.

Additional sewer separation, eliminating as much flow as possible from the large combined sewers would be expected to further reduce at least the BOD load if not both BOD and suspended solids loads to the storm holding lagoon. Such separation would eliminate the low velocity dry weather flows in the combined sewers and in turn the associated sludge banks and other sediment.

Further improvement in the treatment system would be expected if efforts were taken to remove existing sources of storm water, such as downspouts and basement sump pumps, from the separated sanitary system. Although sewer use ordinances now in effect prohibit downspout, footing tile, and sump pump connections, these sources were not removed when existing services were connected to the new sanitary lines. The extent and effect of these storm water connections is not accurately known, and certainly other sources of storm flow enter the separated sanitary system from dry weather flow bypasses and infiltration, but elimination of these connections would provide less dilution of sanitary sewage and a more concentrated influent to the treatment plant.

Storm Holding and Tertiary Treatment Lagoons - The storm holding lagoon is a very successful treatment unit where land is available and plant capacity is much less than needed for combined sewage treatment. Despite short duration periods of heavy organic loading (230 pounds BOD/acre - 26 g/sq m - for Storm No. 12) no offensive odors were noted. In terms of overall removal efficiency for the two storms together, a total of 1,065 pounds (483 kg) of BOD was applied to the lagoon and only 282 pounds (128 kg) of BOD were pumped to the stabilization lagoon, giving a net removal efficiency of about 73%. Suspended solids removal for the two storms was even better. Of the 5,200 pounds (2,350 kg) of suspended solids applied, only 711 pounds (322 kg) were pumped to the stabilization lagoon for a net removal of 86%.

Several design modifications to the storm holding lagoon are suggested by the performance evaluation. First, additional consideration should be given to the location selected for the lagoon bypass or overflow. As previously described, a bypass weir structure was located at the termination of the combined sewer, slightly ahead of the lagoon, with the bypass weir set for maximum lagoon depth. This choice was justified by first flush considerations with the intention being that all of the run-off from a ten-year storm of 1-hour duration would be contained in the lagoon and any remaining flow would be sufficiently weak in suspended solids and BOD to permit bypassing to the stream without serious pollution, and without washing out the more concentrated lagoon contents. The lagoon would then be pumped down over the next 5 days with effluent undergoing further treatment in the tertiary lagoon system.

This method of operation is successful for isolated storms, but causes difficulty during extended rainy periods. The lagoon will fill to the overflow point after an initial rainfall of sufficient depth has occurred, and subsequent combined sewage is bypassed to the stream. If this subsequent run-off is the result of a continuing storm, the first flush has already been contained and the bypassed flow is relatively weak, but if a second storm occurs before the lagoon can be pumped down to a level to create some storage capacity, the first flush from the second storm may be bypassed.

The data collected for this project was not sufficient to establish a good relation between the magnitude of the first flush and the time between successive run-off periods. Studies clarifying the relationship between run-off intensity, strength, and intervals between successive events could suggest new design criteria for holding lagoons and methods of influent management with controlled bypassing. In the interim, however, this problem could be partially solved by locating the bypass or overflow point at the effluent end of the holding lagoon so that all flows must, at least, pass through the lagoon before being discharged.

Several further modifications in holding lagoon design might be considered where solids accumulation are excessive. The influent and effluent pipes in the Shelbyville lagoon have invert elevations corresponding to the lagoon bottom. At the influent line, solids are accumulating in the pipe, apparently due to a silting effect at the influent area in the lagoon. Whether or not this is a temporary condition that is subject to periodic flushing, is not known for sure, but indications are that the depth of material in the influent line is not decreasing.

A similar problem, although not as severe, is a possibility at the effluent pipe from the lagoon to the wet well of the effluent pump station. When the pumps are operating, there is a marked increase in suspended solids in the pumped effluent when compared to the average lagoon contents. This increase is believed due to solids transport from the lagoon bottom.

The effluent silt problem should be easy to overcome by use of an effluent structure in place of the pipe and headwall design. A typical effluent structure would be equipped with a removable scum baffle and either stop planks or other adjustable means of withdrawing effluent from a pre-selected depth in the lagoon.

Preventing silt accumulations at the influent pipe may not be necessary if periodic flows are of sufficient velocity to scour the line. If this is not the case, an alternate method may require consideration, such as elevating the invert of the influent pipe several feet above the lagoon bottom.

Lagoon maintenance is somewhat of a problem since the water surface may vary irregularly by as much as seven feet (2.13 m). Vegetation is difficult to establish on the berms under these conditions and soil erosion must be constantly monitored.

The tertiary lagoon system, judging from the dry weather data, is providing good reductions in nutrients, although BOD and suspended solids are frequently increased due to algae growth. The additional loading to the stabilization lagoon during periods of storm lagoon effluent pumping appear to have very little effect on the tertiary treatment process. Whether or not the polishing lagoon is being overloaded is subject to question, since a large portion of the applied BOD is likely due to algae and as such is not exerted. Dissolved oxygen levels in the tertiary lagoons have always measured close to saturation, and effluent quality is similar to values frequently quoted in the literature for similar facilities.

Southeast Site - The lagoon constructed at the Southeast Site is an example of taking advantage of natural topography. An earthen dam was constructed at the outlet of a natural ravine and the ravine was graded to form an irregular shaped lagoon.

The efficiency of the lagoon as a treatment unit cannot be computed directly because several storms occurred during the wet weather sampling period which could not be monitored. In terms of effluent quality, however, final BOD avlues were consistantly less than 5 mg/l except during storm flows when they were briefly higher. Suspended solids in the effluent averaged higher than what might have been expected, due in part to the presence of finely divided clays washed into the lagoon from the heavily eroded upper reaches of the ravine.

One problem experienced at the southeast lagoon was the sharp increase in effluent BOD and suspended solids during periods of high flow at the influent. Contributing to this problem was the fact that the lagoon was the outlet for the natural drainage in the ravine and influent flows during peak run-off periods were considerably enhanced by natural drainage from the surrounding side slopes. This combined effect produced peak influent flows in excess of 15,000 gpm (945 l/sec) resulting in a direct carry-over of influent BOD and suspended solids to the effluent structure. After the peak influent flow had subsided, effluent BOD and suspended solids quickly dropped to much lower values.

Changing the natural drainage in the ravine so that overland run-off was channeled around the lagoon would have corrected this problem, but the steep natural side slopes would have made this a difficult task, requiring considerable excavation, including construction of an influent berm to route natural surface drainage to either side of the lagoon to drainage ditches of considerable size.

The sluice gate in the lagoon effluent structure was set to an opening such that a completely full lagoon would drain from the eight foot (2.44 m) depth to the normal two foot (0.61 m) depth in five days. This arrangement proved successful except when the lagoon had reached the two foot (0.61 m) depth and the opening in the sluice gate became fouled with floating debris. A scum baffle could be used to prevent this occurrence.

Wet weather flow into a full lagoon overflowed the effluent structure such that lagoon depth never exceeded eight feet by more than several inches. Providing the overflow at the effluent, however, allowed for considerable dilution of the influent flow and resulted in a much weaker discharge. For example, during Storm No. 6, the lagoon was full to the 5.8 foot (1.77 m) level and quickly filled completely. The influent BOD peak measured 190 mg/l, but the effluent only reached 8 mg/l.

The source of the dry weather flow into the lagoon could probably be eliminated by a conscientious sewer inspection and maintenance program, however, the weak flows reaching the lagoon in dry weather probably aid the treatment process by helping to maintain an active biological population between wet weather periods.

The southeast lagoon is really a specialized treatment unit with only a limited application. It was designed to handle high flow rates at infrequent intervals and was actually one of the few treatment alternatives that could economically be used. Combined sewage pumping would have been prohibitively expensive, as would a sewer separation project in the central business district. Other combined sewage treatment alternatives were limited by available land.

Single cell lagoons appear more suited to smaller areas where combined sewage strength is not excessive. The dilution afforded by the lagoon is apparently as much a part of the treatment process as sedimentation and biological activity. Maximum BOD and suspended solids reductions in the lagoon effluents were observed to occur within two days after wet weather flows, although suspended solids continued to decrease gradually over the five-day period. This suggests that the lagoon effluent flow rate may be increased to lower the level more quickly if required. Maintenance of the lagoon is minimal. About the only requirement is weed cutting and general upkeep, although soil erosion must occasionally be checked on the side slopes.

East Site. The storm water treatment tank located at the east side of Shelbyville is uniquely suited for treatment of moderate amounts of combined sewage where pumping costs would be excessive, and available land is limited. The use of a treatment tank, however, required careful consideration of expected quantities of combined sewage, and provision for regulating or limiting flow to the tank to prevent hydraulic overloadings.

The performance of the system during the two storms that were sampled was typical of clarifier units of similar design. BOD removals for the two storms averaged 61% and 65.5% on a pounds removed basis. Suspended solids removal was 53% for both storms. The cummulative BOD and suspended solids for both storms were similar in magnitude, although the first flush characteristics of the second storm were more pronounced, apparently due to the more intense run-off.

Flow regulation at the influent to the treatment tank was provided by a sharp crested weir set at an elevation such that at the maximum hydraulic capacity of the influent sewer, only 1100 gpm (69 l/sec) of flow would pass through the Parshall flume into the treatment tank. Using a 12 foot (3.66 m) bypass weir and 6 inch (15.2 cm) Parshall flume, the hydraulics of the system were such that flows up to 590 gpm (37 l/sec) would enter the tank with no bypassing, but higher flows were subject to increasing amounts of bypassing until at a maximum flow to the tank of 1100 gpm (69 l/sec), bypass quantities reach 3960 gpm (250 l/sec).

While it is obviously necessary to limit flow to the treatment tank to prevent hydraulic overloading, there is some disadvantage in doing so at the tank influent with the bypass weir arrangement. In both the storms sampled during the wet weather evaluation program, peak hydraulic loads came at the beginning of the run-off period during the first flush of BOD and suspended solids. It was also during this time that peak bypass flows were released with chlorination as the only treatment. Since the treatment tank is empty during these initial peak flows it would be more desirable to channel all the initial flow to the tank and only bypass after the tank has filled, thus containing and treating the greatest portion of the higher strength combined sewage.

A possible alternative design could be considered and evaluated. One approach might be to provide a bypass sytem that begins to operate only after the tank has filled. Such a system would be more complicated and require larger influent channels and possibly a comminutor bypass channel for the high flows. The advantages of such a proposed system would have to be weighed against the additional cost and complexity of the treatment unit and the improvement in effluent quality that could be expected.

A second design alternative that warrants consideration concerns the removal of treatment tank underflow. In the Shelbyville system, underflow is controlled by a manually operated valve which remains closed except when draining a full tank. The underflow pipe empties into a manhole in the influent sewer to the pump station wet well, allowing the tank contents and accumulated solids to be pumped into the gravity sewer system leading to the treatment plant. As mentioned previously, a modulating plug valve is located just ahead of the wet well to automatically control the flow of sewage and maintain a constant wet well level during periods of high flow. A possible modification to be considered would entail the installation of two automatic valves, one on the treatment tank underflow line and one on the normal dry weather flow influent line ahead of the junction with the underflow pipe. During dry weather periods, the underflow valve would be closed, and the dry weather influent line valve would open. During wet weather, when flow is entering the treatment tank, the dry weather valve would close and the underflow valve would open. With this type of arrangement, there would be a continuous withdrawal of more concentrated material from the treatment tank rather than a flow of more dilute combined sewage. When flow to the treatment tank ceased, the normal dry weather flow pattern would be resumed.

APPENDICES

Page No.

I. PRE-CONSTRUCTION SAMPLING PROGRAM 91

II. BIOLOGICAL STREAM INVESTIGATION OF THE UNNAMED STREAM SOUTHWEST OF SHELBYVILLE, ILLINOIS 101

III. CONSTRUCTION OPERATION AND MAINTENANCE COSTS 111

APPENDIX I

PRE-CONSTRUCTION SAMPLING PROGRAM

DRY WEATHER FLOW INVESTIGATION

Three sampling stations were erected on the unnamed branch of Robinson Creek adjacent to the Sewage Treatment Plant, and one gauging station established on Robinson Creek. These may be located on the Shelbyville 15 minute quadrangle map (U.S. Geol. Surv. 1942 ed.) in T. 11 N., R. 3 E. as follows: (See Figure 29)

Station A.	Section 13, at the effluent pipe from the existing sewage treatment plant.
Station B.	Section 23, where the stream crosses the line between sections 23 and 24.
Station C.	Section 26, where the stream crosses the line between sections 23 and 26.
Station D.	Section 27, on Robinson Creek 300 feet below the bridge located where the creek crosses the line between sections 22 and 27. This was a stream gauging station and was not part of the 72-hour sampling program.

At sampling stations A, B, and C the stream flow was diverted over a sharp-crested rectangular weir for measurement of the flow.

A Leupold Stevens stage recorder was mounted at each sampling station to provide continuous recording of the flow and to serve as a check on the hourly weir readings.

Station D on Robinson Creek did not involve a fixed structure. This station was chosen as an accessible portion of the creek above the confluence of Robinson Creek with the unnamed branch receiving plant effluent. A grab sample was taken at this station and the creek flow was measured by standard U.S.G.S. techniques described herein.

After construction of the sampling stations on November 18-19, 1968 the 72-hour sampling program began at 4:00 P.M. on November 19, 1968. Hourly samples and measurements were taken on the hour at Station A, 20 minutes after the hour at Station B, and 20 minutes before the hour at Station C. No rain occurred during the 72-hour sampling period and the storm by-pass outfall was dry.

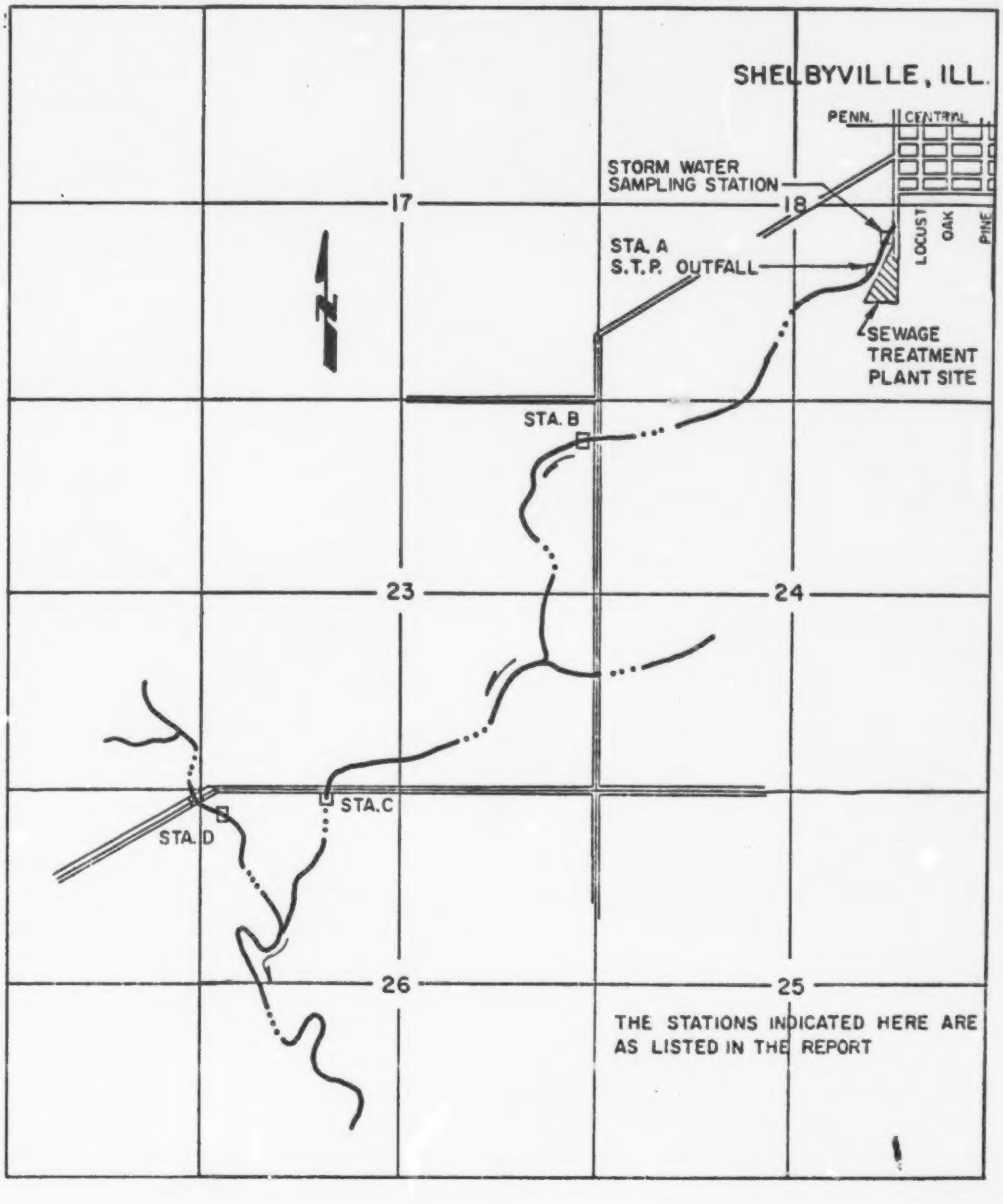

SHELBYVILLE, ILL.
PENN. CENTRAL
STORM WATER SAMPLING STATION
17
18
STA. A S.T.P. OUTFALL
LOCUST
OAK
PINE
N
SEWAGE TREATMENT PLANT SITE
STA. B
23
24
STA. C
STA. D
26
25
THE STATIONS INDICATED HERE ARE AS LISTED IN THE REPORT

Figure 29. Sampling Station Locations

At each station the depth of water over the weir was recorded, the stage recorder chart was marked, a one pint sample was taken and put on ice, and the water temperature and dissolved oxygen were recorded using a Yellow Springs Instrument Model 51 portable dissolved oxygen meter. After 25 samples (24 hours) were collected they were composited in proportion to flow into one gallon containers. Half of the 25th sample was saved to serve as the first sample for the next 24-hour composite. All samples and composites were kept on ice prior to delivery to the testing laboratory.

Except where noted otherwise or where not applicable all analyses were conducted in accordance with the *Standard Methods For the Examination of Water and Waste Water, 12th Edition*. Dissolved oxygen at the time of sampling was obtained by a portable dissolved oxygen meter calibrated in the field. Total coliform was measured by a modified field test method. Millipore Filter field test kits were used to test the composite samples; however, the magnitude of the coliform count at these three stations necessitated dilutions of 10^3, 10^4, and 10^5, in order to obtain less than 200 colonies per filter.

The measurement of the Robinson Creek flow was accomplished using the standard U.S.G.S. stream gauging techniques. The stream cross section was divided into nine, two-foot (0.61 meter) wide intervals (total 18 feet/5.48 meters) and the flow recorded by an ottmeter at the 0.6 depth, or 0.2 and 0.8 depth if depth was greater than 2 feet (0.61 meter). The relationship between flow in the stream and revolutions per minute on the ottmeter is given by the following:

Velocity (in feet per sec.) = 0.8357 n + 0.043 if n less than 1.39
(1 fps = 1.0973 Km per hr.) = 0.8594 n + 0.010 if n greater than 1.39

where n = revolutions per second

Q (flow in cubic feet per sec.) = AV where A = area in sq.ft.
V = velocity in ft. per second.
(1 cfs = 28.317 liters/sec.)

During dry weather conditions the sewage treatment plant is the only apparent source of flow for the unnamed stream in the area adjacent to the treatment plant. It was observed that the flow in this unnamed stream had increased at Station B and then decreased at Station C. This pattern was consistent even for the hourly flow readings. A probable explanation is that field tile between Station A and Station

B are contributing between 50 to 100 gpm (189.3 to 378.5 liter/min) to the stream flow, and between Station B and Station C the water is percolating into a known porous strata.

Physical and chemical data for the three stations and the three days of sampling are given in Tables 14 through 17 and Figure 30.

The dry weather flow investigation of the unnamed stream adjacent to the sewage treatment plant serves several purposes. The general character of the stream environ and water quality gives a basis of comparison to wet weather conditions and to post-construction dry weather conditions. Included in these findings is the type and degree of improvement of environ and water quality as the distance from the treatment plant increases and as the stream approaches the confluence point with Robinson Creek.

The data show that there is improvement in the water quality of the stream at the two stations downstream from the treatment plant. Since the waters of this unnamed stream are probably percolating into the water-bearing aquifer above Station C and since the treatment plant is the only apparent source of dry-weather flow in this stream, the quality of water therein is most important. As will be shown in the section on wet weather flow another pollution problem centers around the combined sewer bypass flows.

On the last day of sampling, the flow of the unnamed stream into Robinson Creek was being diluted at a 25:1 ratio. On this day the Robinson Creek sample showed a BOD load of about 2.3 mg/l. Using the average BOD load in the unnamed stream at Station C of 3.4 mg/l for the same day, this 25:1 dilution would result in a BOD load of 2.6 mg/l in Robinson Creek below the confluence. Robinson Creek serves a tributary area to the west of Shelbyville and as far as is known there are no municipal or industrial waste flows tributary to the creek.

WET WEATHER FLOW INVESTIGATION

The combined storm water and sewage sampling station was erected at the existing 4 x 5 foot (1.22 x 1.53 meter) box sewer outfall near the existing sewage treatment plant. This sampling station may be located on the Shelbyville 15 minute quadrangle map (U.S. Geol. Surv. 1942 ed.) in Section 18, T. 11 N., R. 3 E. See Figure 29.

TABLE 14

DAY 1 - DRY WEATHER SAMPLING DATA

Nov. 19 - Nov. 20, 1968
4:00 P.M. - 4:00 P.M.

	STATION A PLANT EFFLUENT	STATION B VANDALIA RD. BRIDGE	STATION C CONCRETE BRIDGE
Avg.Water Temp.	10.9° C	5.0° C	2.4° C
Avg. Flow	437 gpm (1,655 l/min)	511 gpm (1,930 l/min)	464 gpm (1,755 l/min)
Avg. Dissolved Oxygen by meter at time of sampling	6.6 mg/l	6.5 mg/l	8.1 mg/l
(Composite Tests)			
pH (meter)	8.3	8.3	8.3
Coliform	*approx. 200,000/100 ml	*approx. 30,000.100 ml	*approx. 25,000/100 ml
Settleable Solids	1.0 ml/l	1.0 ml/l	0.7 ml/l
Biochemical Oxygen Demand	11.3 mg/l	7.1 mg/l	9.0 mg/l
Chemical Oxygen Demand	68.0 mg/l	40.0 mg/l	36.0 mg/l
Total Suspended Matter	70.0 mg/l	132.0 mg/l	156.0 mg/l
Volatile Susp. Matter	22.9%	30.0%	34.6%
Residue on Evaporation	817 mg/l	832 mg/l	651 mg/l
Total Volatile Residue	38.6%	33.8%	34.8%

* Modified Field Test

TABLE 15

DAY 2 - DRY WEATHER SAMPLING DATA

Nov. 20 - Nov. 21, 1968
4:00 P.M. - 4:00 P.M.

	STATION A PLANT EFFLUENT	STATION B VANDALIA RD. BRIDGE	STATION C CONCRETE BRIDGE
Avg. Water Temp.	10.7° C	5.5° C	2.9° C
Avg. Flow	404 gpm (1,530 l/min)	466 gpm (1,760 l/min)	426 gpm (1,610 l/min)
Avg. Dissolved Oxygen by meter at time of sampling	6.6 mg/l	6.3 mg/l	8.6 mg/l
(Composite Tests)			
pH (meter)	8.35	8.20	8.40
Coliform	*approx. 155,000/100 ml	*approx. 25,000/100 ml	*approx. 20,000/100 ml
Settleable Solids	1.0 ml/l	1.5 ml/l	1.0 ml/l
Biochemical Oxygen Demand	21.0 mg/l	3.7 mg/l	3.5 mg/l
Chemical Oxygen Demand	80 mg/l	40 mg/l	36 mg/l
Total Suspended Matter	108 mg/l	74 mg/l	122 mg/l
Volatile Susp. Matter	40.7%	54.0%	37.7%
Residue on Evaporation	807 mg/l	820 mg/l	801 mg/l
Total Volatile Residue	46.2%	37.6%	32.8%

*Modified Field Test

TABLE 16

DAY 3 - DRY WEATHER SAMPLING DATA

Nov. 21 - Nov. 22, 1968
4:00 P.M. - 4:00 P.M.

	STATION A PLANT EFFLUENT	STATION B VANDALIA RD. BRIDGE	STATION C CONCRETE BRIDGE
Avg. Water Temp.	12.1° C	8.6° C	5.7° C
Avg. Flow	345 gpm (1,300 l/min)	424 gpm (1,600 l/min)	396 gpm (1,500 l/min)
Avg. Dissolved Oxygen by meter at time of sampling	7.5 mg/l	6.6 mg/l	8.7 mg/l
(Composite Tests)			
pH (meter)	8.2	8.05	8.5
Coliform	*approx. 320,000/100 ml	*approx. 30,000/100 ml	no results available
Settleable Solids	1.0 ml/l	1.5 ml/l	1.0 ml/l
Biochemical Oxygen Demand	28 mg/l	7.3 mg/l	3.4 mg/l
Chemical Oxygen Demand	68 mg/l	45 mg/l	32 mg/l
Total Suspended Matter	122 mg/l	158 mg/l	162 mg/l
Volatile Susp. Matter	36.0%	46.8%	39.5%
Residue on Evaporation	768 mg/l	786 mg/l	810 mg/l
Total Volatile Residue	45.1%	36.6%	28.6%

*Modified Field Test

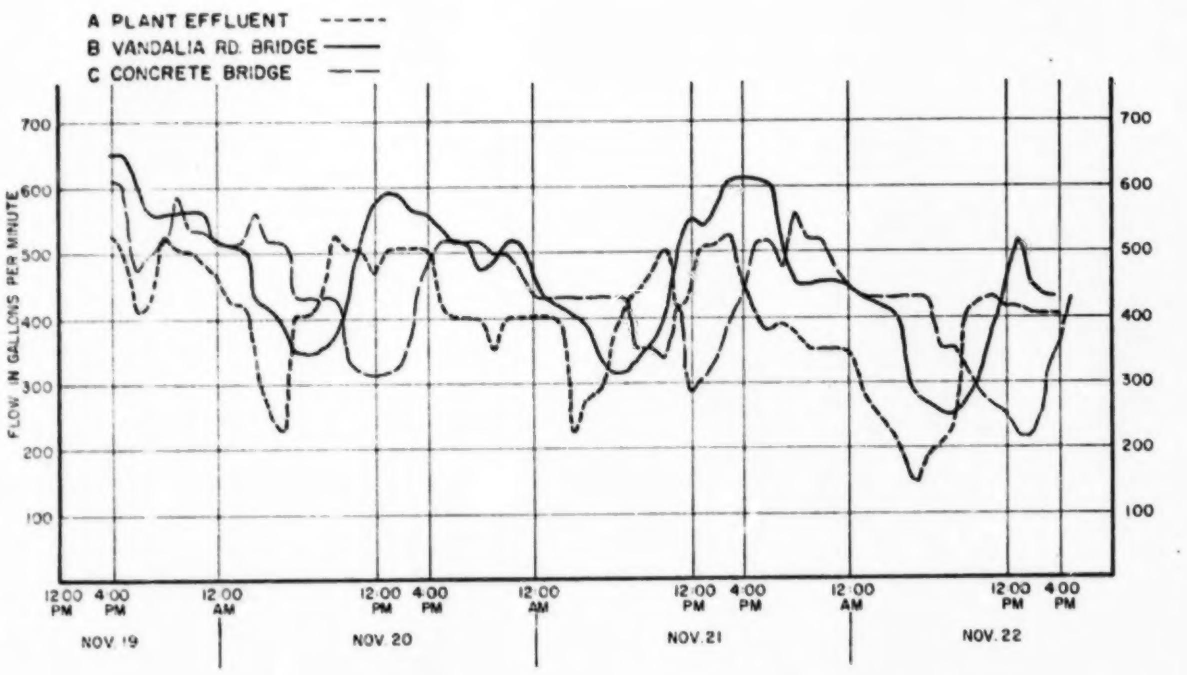
TABLE 17

ROBINSON CREEK GRAB SAMPLE DATA

Nov. 21, 1968

	ROBINSON CREEK STATION D
Water Temp.	8.3° C
Flow	9,930 gpm (37,600 l/min)
Dissolved Oxygen (meter)	8.0 mg/l
pH (meter)	7.95
Coliform	*approx. 10,000/100 ml
Settleable Solids	1.0 ml/l
Biochemical Oxygen Demand	2.3 mg/l
Chemical Oxygen Demand	8.0 mg/l
Total Suspended Matter	130 mg/l
Volatile Susp. Matter	24.6%
Residue on Evaporation	385 mg/l
Total Volatile Residue	55.6%

*Modified Field Test



3 DAY FLOW CHART FOR SAMPLING STATIONS (FIGURE 30)

A box structure with a 48-inch (122 cm) sharp-crested rectangular weir with modified end contractions was cemented into the end of the existing 4 x 5 foot box sewer. A Leupold Stevens 8-day rainfall and flow recorder was mounted at the storm sampling station.

When a large rain storm started the sewage treatment plant operator began sampling as soon as possible and notified the Engineers. At half-hour intervals the weir reading was recorded, a one-quart sample was taken and refrigerated, and the water temperature and dissolved oxygen were recorded using a Yellow Springs Instrument Model 51 dissolved oxygen meter.

Analysis of the half-hour samples were performed in accordance with Standard Methods, 12 Edition. The samples taken at the latter part of each storm were composited into samples of one-hour intervals. Total coliform was measured by the modified field test. Millipore Filter field test kits were used, but the magnitude of the coliform count necessitated dilution of 10^3, 10^4, and 10^5 in order to obtain less than 200 colonies per filter.

APPENDIX II

BIOLOGICAL STREAM INVESTIGATION OF THE UNNAMED STREAM SOUTHWEST OF SHELBYVILLE, ILLINOIS

A series of six stations were examined on the unnamed stream flowing southwest from Shelbyville, Illinois. These may be located in T. 11 N., R. 3 E. on the Shelbyville 15 minute quadrangle map (U.S. Geol. Surv., 1942 ed.) as follows:

Station 1.	Section 13, 200 feet (61 meter) below the storm drain outlet from Shelbyville.
Station 2.	Section 13, 50 feet (15.3 meter) below the present sewage treatment plant outlet.
Station 3.	Section 13. The quadrangle map indicates two houses on the south side of the road which cuts diagonally (NE-SW) across section 13. This station is immediately south of the northeasternmost of these two houses.
Station 4.	Where the stream crosses the line between sections 13 and 24.
Station 5.	Section 24 where the stream crosses the line between sections 23 and 24.
Station 6.	Section 26 where the stream crosses the line between sections 23 and 26.

The course of the stream flows through three general vegetation zones. Station 1 is located in an area offering little cover to the stream. The surrounding vegetation in this area is ragweed (Ambrosia), knotweed (Polygonum) and willow (Salix). Much of the stream in this region is exposed to direct sunlight for most of the day. The stream bottom consists of rocks and gravel and the stream has a moderate rate of flow.

Stations 2, 3, and 4 are located in the wooded portions of the stream. The flora here consists of ragweed (Ambrosia), knotweed (Polygonum), willow (Salix), mulberry (Morus), elm (Ulmus), walnut (Juglans), and poison ivy (Rhus). These plants completely shade the stream from sunlight. Benthic materials consist of silt, sand, and gravel. Water velocity is low in many portions of the stream.

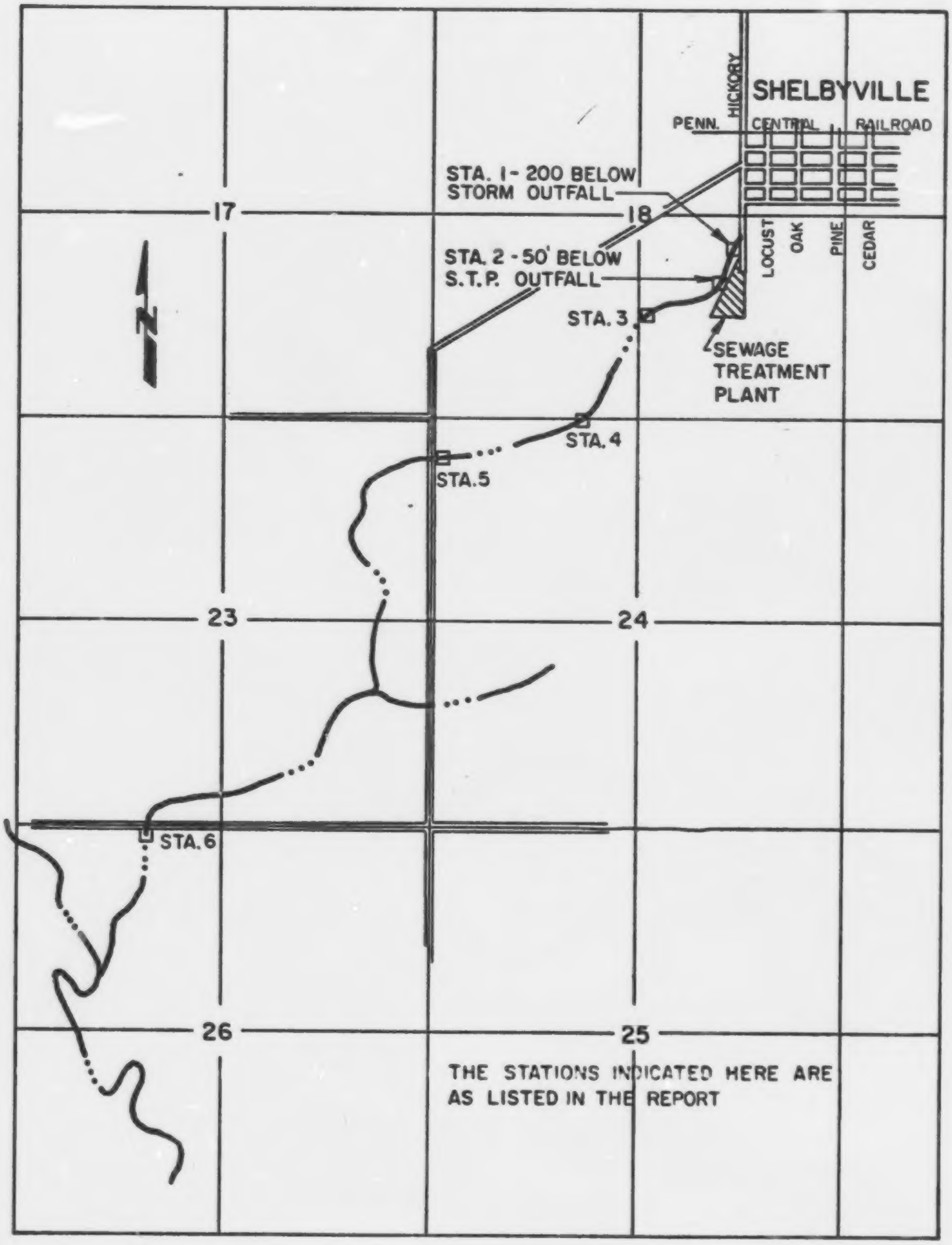

SHELBYVILLE
HICKORY
PENN. CENTRAL RAILROAD
STA. 1 - 200 BELOW STORM OUTFALL
STA. 2 - 50' BELOW S.T.P. OUTFALL
STA. 3
SEWAGE TREATMENT PLANT
LOCUST
OAK
PINE
CEDAR
N
17
18
STA. 4
STA. 5
23
24
STA. 6
26
25
THE STATIONS INDICATED HERE ARE AS LISTED IN THE REPORT

Figure 31. Sampling Station Locations for the Biological Stream Investigation

Stations 5 and 6 are again in areas free from stream cover. Ragweed (Ambrosia), goldenrod (Solidago), and corn (Zea) are represented in the shore flora. Sand and gravel predominate in the benthos and water velocities are moderate.

Benthic organisms were collected using a six-inch (15.24 cm) square Ekman dredge. The Aufwuchs community attached to the substrate was included with the benthos. Samples were sorted in the field and all organisms retained by a 30-meshes-per-inch screen were preserved. Identification and enumeration of organisms was done in the laboratory using a stereoscopic microscope having magnifications to 100 X.

Several physical and chemical parameters were measured in the field. Dissolved oxygen and temperature were measured using a model 54 oxygen meter (Yellow Springs Instruments) calibrated in the field. Hydrogen ion concentrations were measured with a portable expanded scale pH meter model PB (E. H. Sargent). Specific conductivity was measured using a model RA-2A conductivity meter (Beckman). The concentration of total dissolved solids was calculated from specific conductivity and temperature readings. These calculations were run on an IBM 360 data processing system.

Water samples were collected, preserved, and returned to the laboratory for further physical and chemical analyses. The methods set forth in the 12th edition of Standard Methods were followed. Turbidity measurements were made using a Hach model 2100 turbidimeter. Colorimetric tests were run with the aid of a Bausch and Lomb Spectronic 20 spectrophotometer.

In addition to the macroinvertebrates encountered, several vertegrate species were observed. These include the muskrat, Ondatra zibethica, and several species of adult frogs (Rana catesbeiana, R. palustris, R. pipiens, and Acris crepitans). No fishes were observed in the stream.

Invertebrate organisms recorded in and around the stream are recorded in Table 18. The classification of these organisms has been expanded to clarify their identification for the non-biologist.

Table 19 represents quantitative data regarding the occurrence of the various organisms at the six biological sampling stations. A plus mark (+) indicates the presence of an organism which was not taken by quantitative means.

Physical and chemical data from the three physical-chemical stations are given in Table 20. Minimal data from the "biological only" stations are also included.

Table 18 List of the organisms found in and around the unnamed stream near Shelbyville.

Phylum	Class	Order	Family	Genus-Species	Common Name
Mollusca	Gastropoda	Pulmonata	Physidae	*Physa*	Pouch snail
Annelida	Oligochaeta	----	Tubificidae	*Tubifex*	Sludge worm
Annelida	Oligochaeta	----	Lumbricidae	*Eisenia foetoda*	Earthworm
Annelida	Hirudinae	----	----	----	Leech
Arthropoda	Crustacea	Isopoda	----	----	Sowbug
Arthropoda	Insecta	Collembola	Poduridae	*Podura equatica*	Springtail
Arthropoda	Insecta	Hemiptera	Saldidae	----	Shorebug
Arthropoda	Insecta	Coleoptera	Dytiscidae	*Laccophilus*	Predaceous diving beetle
Arthropoda	Insecta	Coleoptera	Staphilinidae	*Carpelimus*	Rove beetle
Arthropoda	Insecta	Coleoptera	Carabidae	*Bembidion*	Ground beetle
Arthropoda	Insecta	Coleoptera	Gyrinidae	*Dineutus*	Whirligig beetle
Arthropoda	Insecta	Coleoptera	Hydrophilidae	*Tropisternus*	Water scavenger beetle
Arthropoda	Insecta	Coleoptera	Heteroceridae	*Heterocerus*	Varigated mud-loving beetle
Arthropoda	Insecta	Diptera	Culicidae	*Culex*	Mosquito
Arthropoda	Insecta	Diptera	Chironomidae	----	Bloodworm,Midge
Arthropoda	Insecta	Diptera	Syrphidae	*Tubifera*	Rat-tailed maggot
Arthropoda	Insecta	Diptera	Psychodidae	*Psychoda*	Filter fly
Arthropoda	Insecta	Diptera	Muscoid-type	----	Maggot

Table 19 Abundance of the various taxa at the six biological stations in the unnamed stream near Shelbyfille,Ill.

Taxa	Station 1	2	3	4	5	6
Laccophilus adults	+	14			+	
Culicidae pupae	+	29	14	14	+	
Culicidae larvae	+	100	+	+	+	
Chironomidae larvae	+	14	29	14	3241	6238
Tubifex	+	144		516	229	287
Eisenia foetoda	+		+			
Physa	+		14			+
Salididae(Terrestrial)	+				+	
Podura aquatica	+				14	
isopoda(Terrestrial)		14				
Tubifera larvae		301				
Psychodidae pupae		29	57	14		
Psychoda larvae			129	14		
Carpelimus(Terrestrial)			+		+	
Bembidion(Terrestrial)			+		+	+
Tubifera pupae				+		
Hirudinea cocoons				14		
Muscoidae larvae				+	+	
Dineutus adults					+	
Heterocerus adults (Terr.)					14	
Chironomidae pupae					272	
Tropisternus adults						+

Table 20 Physical and chemical data from six stations on the unnamed stream near Shelbyville (mg/l unless otherwise indicated).

Factor	Station 1	2	3	4	5	6
Temperature	24.8	20.9	20.5	21.6	18.8	22.5
pH	7.21	6.78	6.99	6.90	6.75	7.35
Turbidity (JTU)		7			5	4
Conductivity (umhos/cm)	1050	1000	1050	900	1225	950
COD		92.06			61.73	57.14
O_2	0.65	4.45	2.52	2.40	1.40	5.78
CO_2		90			110	34
SiO_2		15.3			14.7	15.0
Total Dissolved solids	766	795	840	706	1014	731
Cl^-		66.04			139.36	47.56
NO_3^-nitrogen		0.95			0.25	1.00
NO_2^-nitrogen		0.228			0.218	0.380
OH^-(as $CaCO_3$)		0.0			0.0	0.0

(Continued on next page)

Table 20 continued

Factor	Station 1	2	3	4	5	6
HCO_3^- (as $CaCO_3$)		309			321	292
CO_3^{-2} (as $CaCO_3$)		0.0			0.0	0.0
SO_4^{-2}		10.5			5.2	3.2
SO_3^{-2}		1.0			1.0	1.0
PO_4^{-3} ortho		57.5			40.5	46.5
PO_4^{-3} filtered poly		1.0			0.5	9.0
PO_4^{-3} total		58.5			41.0	55.5
Ca		32.0			39.8	28.9
Mg		56.3			48.3	56.3
Fe		0.52			0.50	0.62
Hardness (Ca,Mg,Fe as $CaCO_3$)		312.5			299.1	305.0

Included in the list of organisms present in and around the stream (Tables 19 and 20) are several groups of terrestrial organisms normally occurring near water. This distribution is not related to the quality of the water since they only seek moist situations. These organisms include:

Eisenia foetoda
Isopoda
Saldidae
Carpelimus
Bembidion
Heterocerus

The remaining organisms are generally considered as truly aquatic, but in a way this is a misleading statement. The beetles of the genera *Laccophilus*, *Dineutus*, and *Tropisternus*, the Collembolan, *Podura aquatica*, and the snail, *Physa* derive their oxygen directly from the atmosphere and are not indicative of water quality.

The flies of the families Psychodidae, Syrphidae, Culicidae, and Chironomidae, and the sludge worm, *Tubifex* do provide an indication of stream conditions. In the most septic portions of the stream large numbers of *Tubifera*, *Psychoda*, and *Culex* are present. These organisms also obtain their oxygen directly from the atmosphere but are generally thought of as pollution indicators. *Psychoda* is tolerant enough to breed extensively in trickling filters of sewage treatment plants.

Tubifex and the chironomid larvae derive their oxygen from the water and are therefore not expected to occur where it is absent. *Tubifex* is the most tolerant of these organisms. It is expected that they will occur where chironomids are unable to survive. The lower lethal limit of oxygen for either of these organisms, however, is far below any value encountered at Shelbyville.

The downstream replacement of organisms, from more tolerant to less tolerant species, is best illustrated graphically (Figure 32). This pattern is certainly not the expected result when the dissolved oxygen levels are considered. Rather it represents a general improvement of the entire stream downstream from the treatment plant. Most important here would be the disappearance of the anaerobic sludge deposits on the stream bed.

The physical and chemical data indicate that the stream is subject to periodic "slugs" of dissolved substances in the treatment effluent. The sharp fluctuations in total dissolved

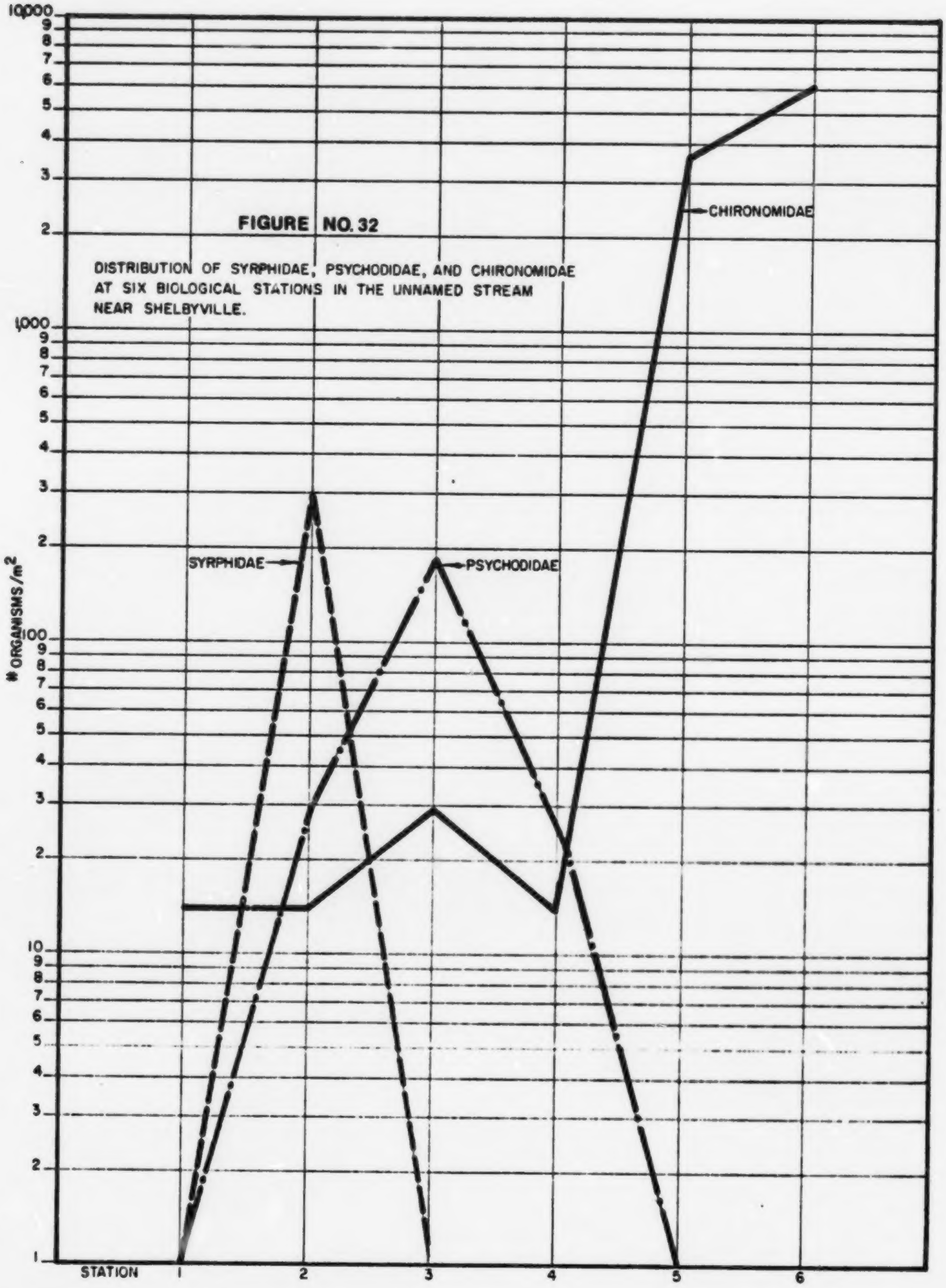


FIGURE NO. 32

DISTRIBUTION OF SYRPHIDAE, PSYCHODIDAE, AND CHIRONOMIDAE AT SIX BIOLOGICAL STATIONS IN THE UNNAMED STREAM NEAR SHELBYVILLE.

solids among stations 4, 5, and 6 best illustrate this. The decrease in chemical oxygen demand and dissolved oxygen downstream indicates the oxidation of reduced substances. Hydrogen sulfide, present in the benthos at stations 2, 3, and 4 is probably a major consumer of oxygen. Sulfates, however, do not show an increase downstream. Oxidation of nitrogen could also account for oxygen consumption. The high nitrite value for station 6 probably represents a "slug" of more concentrated effluent.

The majority of the data regarding the physical and chemical nature of the stream is within the limits of tolerance of most organisms.

APPENDIX III

CONSTRUCTION, OPERATION AND MAINTENANCE COSTS

It must be realized that construction costs can vary significantly, depending upon prevailing labor, materials, and geographical conditions. The following information reflects the cost of construction of the three treatment facilities, completed in 1971. Figures were taken from the final pay estimate for the project.

Southeast Site

Site Clearing	$ 3,800
Earthwork	9,000
Access Road	1,800
Fencing	1,400
Effluent Structures	16,900
Piping and Valves	6,900
	$39,800

Southwest Site

Plant Effluent Pump Station Force Main & Related piping	$ 43,000
Site Clearing	4,000
Earthwork (3 lagoons)	46,000
Influent, Effluent & Bypass Structures	27,200
Storm Holding Lagoon Effluent Pump Station	20,000
Chlorination Facilities	56,300
60 in. storm lagoon Influent Line	63,000
Piping	10,700
Metering & Electrical	3,000
Fence, Gates, Access Road Culverts & Seeding	36,300
	$309,500

East Site

Site Clearing	$ 2,500
Pump Station & Force Main	26,800
Treatment Tank	69,800
Chlorination Facilities	14,600
Piping & Valves & Valve Chamber	21,800
Tank Influent Structure	8,200
Electrical	3,500
Fence, Gate, Access Road	6,400
	$153,600

The operation and maintenance costs are not expected to vary significantly with the frequency of usage. The following summary of anticipated labor is an estimate based on mandays, so that any appropriate wage scale can be applied.

	Mandays labor Annually
Southeast Site	
Weed & grass control	8
Berm and Bank maintenance	15
Painting & Structure Upkeep	2
Access Road & Fence repair	2
	27
East Site	
Weed & grass control	2
Mechanical Equipment Maintenance	6
Pump Station Maintenance	30
Chlorination Equipment	12
Fence and Access Road Repair	2
Cleaning & Hosedown	24
	76
Southwest Site	
Weed & grass control	15
Berm & Bank Maintenance	20
Painting & Structure Upkeep	6
Access Road & Fence Repair	2
Plant Effluent Pump Station Maintenance	30
Storm Holding Lagoon Effluent Pump Station Maintenance	5
Chlorination Equipment	24
	102

Together, the operation and maintenance labor for the three sites totals 205 mandays. This figure does not include any allotment for operating and maintaining the secondary treatment plant, and is probably liberal in the time estimated for the various tasks. The minimum operation and maintenance labor required to keep facilities operating properly would probably be about 125 mandays a year.